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D. M. BEACH, *Editor*

Volume 23, No. 1 March 1942

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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THE DESIGN OF ROADSIDE DRAINAGE CHANNELS

BY DISTRICT 10, PUBLIC ROADS ADMINISTRATION

Reported by **CARL F. IZZARD**, Associate Highway Engineer

Good surface drainage is an important element in the safe, convenient, and economical use of a highway. This report presents a procedure for analyzing drainage problems and designing channels to avoid future difficulty and abnormal expense in maintaining the surface drainage system. This procedure consists of first estimating the peak rate of runoff from each drainage area contributing to channels along the highway; second, checking the ability of these channels to carry the estimated discharge without eroding or overflowing; and third, designing protection against erosion or designing modified channel sections for increased capacity where necessary.

This discussion is limited to consideration of the problem in humid sections of the country where sod can be readily established. The "thatching" action of sod in protecting the soil from erosion increases manyfold the depth of water that may be satisfactorily carried in a channel and also increases the maximum gradient permissible for such a channel. The cost of providing sod is usually substantially less than the cost of paved gutters, the design of which is also discussed.

THE SCOPE of this study is the design of roadside drainage channels with respect to erosion control, adequate capacity for normal peak rates of runoff, landscape considerations, efficient maintenance, ultimate economy, and traffic safety. Because climatic conditions greatly affect methods that may be successfully employed for erosion control, the scope is limited geographically to the humid regions where rainfall is sufficient in amount and seasonal distribution to support native grasses forming a good sod.

The primary purpose of the roadside drainage channel is to provide for efficient removal of surface runoff from the roadway so that traffic can move safely and cars can park clear of the paved roadway in rainy weather when necessary. The channel, therefore, should have adequate capacity to carry the normal peak rate of runoff without overflowing onto the road shoulders. For ultimate economy, the channel should retain this capacity during the life of the road without excessive maintenance costs; this means that the channel should not erode or silt up. In the snow belt, the channel may be designed for snow storage. Since safety is of paramount importance in highway design, the drainage channel should have gently sloping sides and a slightly rounded bottom (see the cover illustration) so that a vehicle forced off the roadway can run down into the channel without overturning. In hilly or mountainous country, practical considerations of economy in first cost will limit the extent to which this ideal cross section may be attained. Prevailing standards of right-of-way width should not be permitted to restrict the width of channel necessary for hydraulic capacity or traffic safety.

The modern highway cross sections in use in most of the State highway departments are in accord with these design principles in varying degrees. The emphasis on traffic safety together with the increasing appreciation of good landscape design has resulted in widespread acceptance of the wide, shallow roadside drainage channel in place of the deep, narrow ditch carried over from railroad design. The practice of seeding or otherwise artificially establishing sod on shoulders, slopes, and drainage channels where feasible has developed as the necessity for controlling erosion became apparent. The principal object of this study is to present a simple method of determining in advance of construction the portions of drainage channels that are likely to erode unless treatment other than ordinary seeding is provided.

EROSION HAZARD IN DRAINAGE CHANNELS DETERMINED

Control of erosion is fundamentally a matter of controlling the velocity of surface flow or of treating the material so that it will withstand the velocity likely to occur. The velocity at which water will flow in a given channel depends on the interrelated factors of grade, shape of cross section, roughness of channel lining, and the rate at which water is delivered to the channel. Figure 1 is a simple diagram which takes all these factors into consideration.

This diagram is to be used for investigating the erosion hazard in the typical roadside channel and is applicable only to the cross section for which it is computed. Since simplicity in design and construction requires the use of a standardized cross section as much as possible this limitation is not a serious drawback, particularly since similar diagrams for other shapes of channel commonly used can be readily prepared. Figures 2 and 3 are diagrams for two other cross sections. The curves are computed by the Manning formula (see table 1).

TABLE 1.—*Channel roughness for use in selecting scales on velocity-discharge diagram* [1]

Type of lining	Scale	Value [2] of n
Ordinary earth, smoothly graded	Smooth	0. 02
Jagged rock, or rough rubble	Rough	. 04
Rough concrete	Smooth	. 02
Smooth rubble	do	. 02
Well-maintained grass, depth of flow over 6 inches	Rough	. 04
Well-maintained grass, depth of flow under 6 inches	Very rough	. 06
Heavy grass	do	. 06

[1] In general, use roughest condition likely to exist for estimating capacity, and smoothest condition for estimating velocity.
[2] In the Manning formula.

The use of the diagram may be illustrated as follows: Assume that peak rate of run off has been estimated at 10 cubic feet per second for a channel having the cross section shown in the diagram and a grade of 4 percent. To check the velocity on bare earth, use the "smooth" scale ($n=0.02$); move along the curve for 4 percent grade to point "a" where discharge equals 10 cubic feet per second and read 6.8 feet per second on velocity scale. This velocity is excessive for earth (see table 2) so it is concluded that sodding is necessary. With sod

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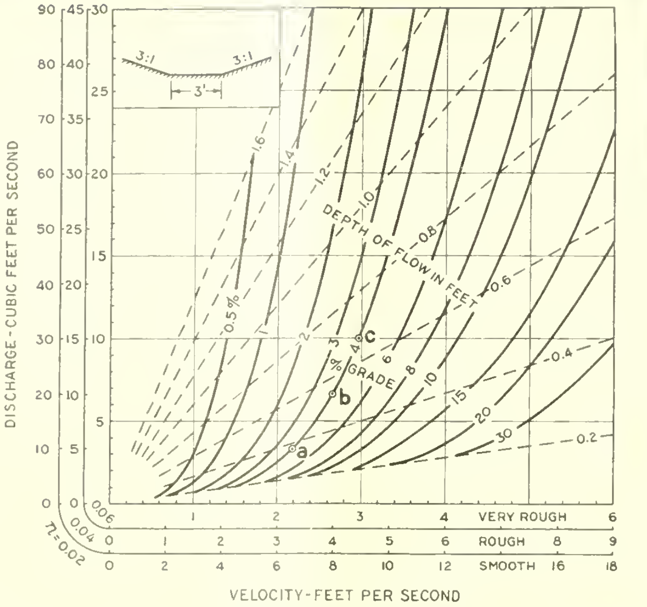

Figure 1.—Relations Between Grade, Depth, Discharge, and Velocity For a Channel 3 Feet Wide at the Bottom and Having 3:1 Side Slopes, For Various Linings.

use the "rough" scale ($n=0.04$); from the curve for 4 percent grade at "b" read velocity equal to 4 feet per second. This velocity is allowable for good sod.

Table 2.—*Allowable velocities in various channels*

Type of lining	Allowable velocity
	Feet per second
Well-established grass on any good soil: [1]	
Bermuda grass	6
Bluegrass	5
Smooth brome grass	5
Western wheat grass	4
Buffalo grass	4
Sudan grass (annual, temporary cover)	3
Common lespedeza (annual, reseeding)	3
Lespedeza sericea	3
Earth without vegetation: [2]	
Fine sand or silt, noncolloidal	1-2
Ordinary firm loam	2-3
Stiff clay, very colloidal	4
Clay and gravel	4
Coarse gravel	4
Shale	5

[1] Data from Roadside Development Report, Part II, Appendix IV, April 1940.

[2] Data adapted from recommendations of Special Committee on Irrigation Research, American Society of Civil Engineers 1926.

If grasses are allowed to grow rank the resistance to flow is increased and the velocity is still further decreased. Since a decrease in velocity must result in an increase in cross-sectional area of flow if the given peak rate of discharge is to be obtained, the channel may be overflowed. To check on this condition the depth of flow should be investigated as follows:

Dotted lines are drawn through points of equal depth of flow on the discharge curves. Thus in the preceding example water would flow about 0.4 foot deep in a bare earth channel (point "a"), or about 0.55 foot deep in the sodded channel (point "b"). If the grasses are allowed to grow rank the flow will be approximately as shown on "very rough" scale ($n=0.06$), point "c," which lies about one-third of the distance from the 0.6 to the 0.8 foot depth. The depth of flow is thus approximately 0.67 foot with rank grasses. This illustrates why the required depth of a channel should be based on the roughest condition of the channel likely to exist. A freeboard of at least 0.3 foot should be provided above this maximum depth of flow to allow for silting, for wave action, and for a factor of safety against too low an estimate of peak runoff.

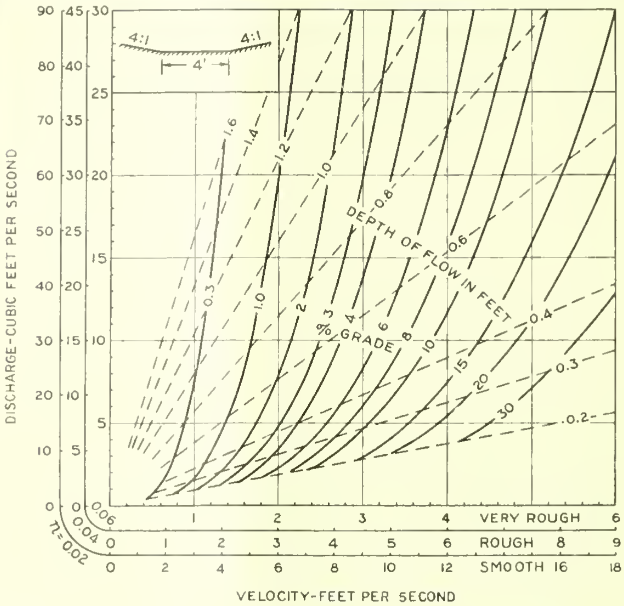

Figure 2.—Relations Between Grade, Depth, Discharge, and Velocity For a Channel 4 Feet Wide At the Bottom and Having 4:1 Side Slopes, For Various Linings.

In channels with comparatively flat side slopes the discharge capacity increases rapidly with small increases in depth, and consequently the allowance of 0.3 foot freeboard is usually adequate. In the example, the discharge capacity at a depth of 1.0 foot is more than double that at a depth of 0.67 foot. The resulting velocity is about 5.5 feet per second on the "rough scale" which is above the recommended limit for bluegrass sod, but since this limit is conservative, no appreciable damage need be anticipated by a runoff double that for which the channel was designed. Silting in the bottom of the channel, while reducing the depth of flow, will reduce both the cross-sectional area and the discharge capacity only a small amount. For the same reason the rounding of the bottom of the channel usually shown on typical cross sections does not significantly affect the discharge capacity computed for a strictly trapezoidal section.

SEVERAL MEANS OF INCREASING CAPACITY

In any channel with fixed side slopes the discharge capacity may be increased by increasing (1) grade, (2) bottom width, (3) depth, or through decreasing resistance to flow by providing a smoother lining. Increasing the grade is frequently impractical. Increasing the bottom width has the least effect on velocity and is therefore desirable where velocity is close to the limit. On the other hand, increasing the depth, while increasing velocity slightly more than that with increased bottom width, is the simplest procedure and also requires less over-all width. The latter consideration is important where right-of-way is restricted or expensive. Smoothing of the channel lining provides increased capacity without increased width (or may even permit a reduction of width) but, since this must generally be accomplished by some kind of paving, the cost may become excessive.

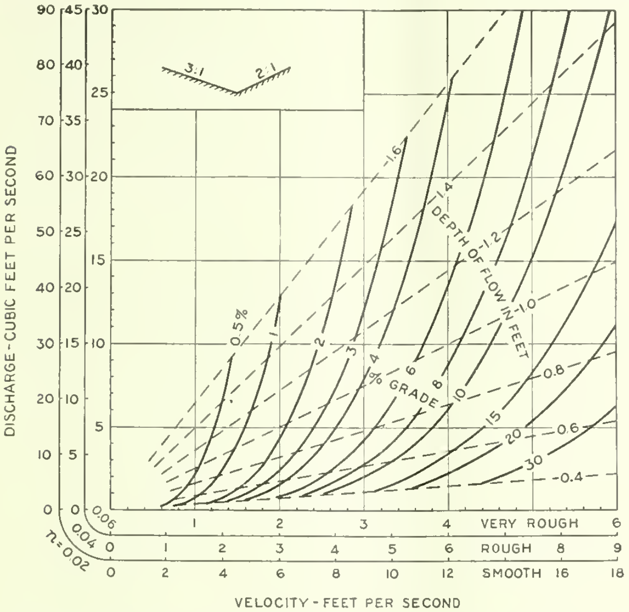

FIGURE 3.—RELATIONS BETWEEN GRADE, DEPTH, DISCHARGE, AND VELOCITY FOR A V-SHAPED CHANNEL HAVING 3:1 AND 2:1 SIDE SLOPES, FOR VARIOUS LININGS.

The effect of channel cross section on depth of flow, velocity, and required width of channel is demonstrated in table 3, based on figures 1, 2, and 3. Note that the V-shaped channel has the greatest depth of flow and requires nearly as much width as the section with a 3-foot flat bottom. The difference between the discharge flowing full and 10 is the reserve capacity afforded by the 0.3-foot freeboard.

Instead of providing additional capacity in a given channel it is sometimes possible to divert part of the runoff into another channel as by the construction of an intercepting ditch on top of the cut slope. Treatment of the drainage area to increase infiltration, by mulching or other means, will also reduce the peak rate of runoff.

TABLE 3.—*Examples of effect of channel shape on capacity* [1]

	3-foot bottom 3 : 1 slopes	4-foot bottom 4 : 1 slopes	V-bottom 3 : 1 and 2 : 1 slopes
Depth of flow feet	0.95	0.80	1.45
Depth plus 0.3 foot freeboard do	1.25	1.10	1.75
Velocity feet per second	1.8	1.6	1.9
Width (including freeboard) feet	10.5	12.8	8.8
Discharge flowing full cubic feet per second	17	18	16

[1] Taken from figures 1, 2 and 3 with discharge of 10 cubic feet per second flowing in sodded channels on a 1-percent grade ($n=0.06$).

On grades of less than 2 percent the design of a sodded channel is likely to be determined by capacity requirements. As the grade is steepened the velocity increases and a point is reached beyond which the maximum allowable velocity becomes the determining factor. For a given rate of discharge and a fixed grade the velocity may be kept within reasonable limits by increasing the bottom width of the channel or by flattening the channel slopes. Both of these methods require a substantial increase in the overall width of the channel. Where it is not feasible to design a sodded channel, it may be paved; velocity is then no longer the governing factor. The capacity of a channel of given shape and depth may be doubled by changing the lining from well-maintained sod to concrete or masonry, or tripled if the initial lining was a rank grass.

Outfall channels carrying water discharged from cut sections frequently involve steep grades which would cause excessive velocities to be developed in ditches of standard cross section. The solution of this problem is to pave the channel or to increase the bottom width. The required bottom width of a sodded channel may be approximated by the following formula

$$b_2=b_1\times\frac{S_2}{S_1}$$

in which b_2 is the required bottom width, b_1 is the width of the standard section, S_2 is the proposed grade and S_1 is the grade on the standard section at which the given discharge produces the maximum allowable velocity. Use of this rule eliminates the necessity of preparing diagrams similar to figures 1 and 2 for channels with increased bottom width.

For traffic safety, side slopes of a paved channel should be as flat as practicable in locations where vehicles might be forced to drive down into the channel. In other locations, as at the top of a cut slope, or the toe of a fill slope protected by guard rail, economy dictates the use of a more efficient hydraulic section. The most efficient section, a semicircle, is difficult to construct and a semihexagonal section is preferable. If high velocities are developed in paved channels, provision must be made for checking the velocity at the outlet unless the channel discharges into a pool of water of appreciable depth. Velocity may be checked by a wide apron with projections or baffle walls on the surface or by a low sill at the end, creating a stilling pool. Current research on this problem, which exists at culvert outlets also, should provide more definite principles for the design of energy-dissipating structures.

WIDE, SHALLOW CHANNELS DESIRABLE

The ideal roadside channel cross section to be built in earth adjacent to an earth or stabilized shoulder should have a slope from the shoulder of at least 4 : 1 (which is the steepest slope that permits a driver in the outside lane to see the entire length of the slope), a rounded bottom at least 4 feet wide, and a back slope not steeper than 4 : 1. The depth should vary from a minimum of about 1 foot below the edge of shoulder in regions of low rainfall intensities to a minimum of 1½ feet in the Southern and Gulf Coast States. Depths and widths should be increased to provide additional capacity where the runoff analysis indicates that peak runoff rates from the tributary drainage areas are of such magnitude that the water is likely to rise above an elevation 0.3 foot below the shoulder line. When the ideal cross section results in an excessive amount of excavation which cannot profitably be used for embankment construction (as in flattening embankment slopes), the width of the bottom may be reduced and the backslope steepened to not more than 2 : 1, provided the discharge capacity remains adequate. The slope from the shoulder may also be increased to a maximum steepness of 3 : 1. The resulting cross section, however, will have very limited capacity and may require closely spaced culverts or auxiliary channels (intercepting channels or storm sewers) to avoid overflowing onto the shoulder during times of peak runoff.

The ideal cross section is based first of all upon obtaining a channel that a vehicle can cross at any angle with a reasonable chance of remaining upright. But this section is also favorable to establishing and maintaining a good sod (it is flat enough to mow mechanically), provides space for snow storage, and lessens the chances of snowdrifts accumulating (because the wind blows across without eddies). Finally, the gentle slopes covered with sod easily merge into the landscape. The spaciousness of the entire roadway cross section tends to relieve the tension of driving because the driver feels that he can safely drive off the pavement in an emergency. Any departure from this ideal cross section that tends to narrow the roadway is usually a compromise between safety and economy or expediency.

In solid rock cuts, the backslope will necessarily be very steep, approaching the vertical in the extreme case. In such instances the drainage channel may have a narrow, rounded, V-shaped cross section just large enough to carry the peak runoff without overflowing on the shoulder. Another design having considerable merit requires paving the entire shoulder and providing for drainage in a monolithic curb and gutter with closely spaced catch basins emptying accumulated drainage into cross culverts or into storm sewers. Such a design balances the cost of additional excavation in a wider cut against the cost of more expensive drainage facilities and at the same time affords a maximum of all-weather traffic safety. It is difficult to maintain narrow sodded shoulders between a paved gutter and the paved roadway; in such cases the shoulder should be stabilized and surface-treated.

Obviously, erosion is not a problem in roadside channels cut in solid rock. However, since it is seldom practical to finish the channel smoothly, it may be advisable to pave the channel with concrete or rubble masonry to permit sufficient velocity so that extra cross-sectional area is not needed for the required capacity. Furthermore, in rock cuts there is usually a continual accumulating of spalled rock, cinders or sand from ice treatment, and loosened earth, which will be more readily removed by water flowing in a smooth-lined channel than in a jagged, rock-lined channel.

DITCH CHECKS AND DROP STRUCTURES SOMETIMES USED

Another method of controlling velocity in channels is to flatten the grade and build a series of steps, with abrupt drops built out of concrete, stone, timber, or other suitable material constructed at each step. The channel between drops is designed for a non-eroding velocity at the peak discharge rate. The structures are designed to discharge through a weir notch, which may have the same shape as the channel above, but is more frequently a constriction in the channel. In order to prevent under-scour an apron with wingwalls must be provided below the drop. The spacing of the drop structures will depend on the general gradient of the channel, the intervening gradient, and the vertical drop at the structure.

If the channel between structures is designed for a non-eroding velocity of about 2 feet per second in bare earth the intervening gradient will be less than 0.5 percent and the number of structures per 100-foot station will be approximately equal to the general gradient in percent divided by the drop in feet at each structure. Thus on a 5 percent grade, structures having a 0.5 foot drop would be spaced 10 to the station. The cost per structure would depend on the material and the size of notch necessary for the peak discharge. Properly designed concrete structures will cost from $5 to $10 or more each, or on a 5 percent grade, from $50 to $100 or more per station. For the same cost per station it would be possible in most cases to construct a paved, continuous gutter 3 to 5 feet wide, or a continuous sodded gutter 9 feet wide, either of which should adequately carry the discharge. The cost per station for drop structures will be less for higher drops but safety considerations limit the height of drop in roadside channels.

Where sod is used to line the channel between drop structures the intervening grade may be increased to as much as 5 percent (determined by reference to a velocity–discharge diagram such as fig. 2) which in most cases eliminates the need for stepping the grade line.

One of the main objections to building drop structures in channels parallel and close to the roadway is the hazard to traffic. The structures also interfere with mowing operations and are always unsightly. In some soils they are difficult to maintain because washing out of the soil occurs below or around the structure. Drop structures are useful, however, in channels normally inaccessible to vehicles on the roadway, as where a channel drops off suddenly into a creek bed. In such cases the drop should be designed as an engineering structure with ample notch capacity and protection against scour at the downstream end.

Any method of designing a channel for a limited velocity and adequate capacity necessitates an estimate of the peak rate of discharge which the channel will be expected to carry. This factor is very important but has received relatively little attention. The grade of the channel is usually thought of as being the criterion upon which the occurrence of erosion must be based, but a glance at figure 1 shows that velocity, which is the principal factor causing erosion, changes rapidly with the rate of discharge, particularly for low flows, regardless of the grade.

RUNOFF ESTIMATED BY SEVERAL METHODS

The peak rate of runoff may be estimated by several methods: The Burkli–Ziegler and the so-called rational formulas, and runoff curves developed by the United States Department of Agriculture. The rational formula is preferred to the Burkli–Ziegler because the slope factor introduced in the latter is of questionable value. The slope should be considered in estimating the time of concentration used in the rational formula by making a rough approximation of the probable velocity in the channels through which the water flows from the most remote part of the drainage area. This may be accomplished with velocity–discharge diagrams, using an assumed discharge. The velocity of overland flow on grass, where flow is not concentrated in a defined channel, may be roughly approximated by the rule of thumb that "the velocity in feet per second is approximately equal to one-tenth the square root of the total fall in feet, provided the grade is not more than 20 percent."

The simplest method of estimating runoff, probably as reliable as any other, is the use of runoff curves of the type published by the Highway Research Board in April 1940, in Appendix IV of the Roadside Development Report, Part II. By selecting a curve representing the topographic, soil, vegetal cover, and surface storage characteristics of the drainage area, the peak rate of discharge may be read directly in cubic feet per second from the size of the area in acres.

(*Continued on p. 13.*)

LAND USE PLANNING IN RELATION TO HIGHWAYS

WITH SPECIFIC REFERENCE TO DELTA COUNTY, COLORADO

Reported by **W. J. KELLER**, Highway Engineer, District 3, Public Roads Administration

THE Colorado Agricultural Land-Use Planning Committee was organized on April 26, 1939. The original group consisted of 12 farmer-rancher members representing the 12 major types of farm areas in Colorado, and one representative from each of the following State and Federal agencies:

U. S. Agricultural Adjustment Administration.
U. S. Bureau of Agricultural Economics.
U. S. Farm Credit Administration.
U. S. Farm Security Administration.
Office of State Forester.
U. S. Public Roads Administration.
U. S. Soil Conservation Service.
State Extension Service.
State Experiment Station.
State Highway Department.
U. S. Forest Service.

Since organization, representatives have been added to the committee from the Taylor Grazing Service, the Fish and Wildlife Service, and the State Planning Commission.

The objectives of the planning program are to develop an agricultural land-use program in which will be correlated the suggestions and work of the farmers and ranchers, the State Agricultural Extension Service and Experiment Station, the Bureau of Agricultural Economics, and operating agencies of the Federal and State governments; and to produce a planning program in which all of the above-mentioned agencies can effectively contribute to agricultural adjustment, conservation, crop insurance, farm forestry, flood control, land retirement, rehabilitation, and water utilization.

As a part of the general program of land conservation and utilization, State and county committees have been appointed in every State to study the problems confronting them intimately. These committees are generally composed of local farmer members and members of the various State and Federal agencies concerned.

The Public Roads Administration has recognized the close relationship between the objectives of land use planning and the problems confronting highway administrators and is cooperating fully in the general program. Representatives attend all State meetings and many of the county meetings, acting in an advisory capacity on matters pertaining to roads, soil erosion, flood control, aerial photography, mapping, etc. The Public Roads Administration has been able to contribute substantially to the general program by furnishing maps and various statistical data compiled by the State-wide highway planning surveys.

Under the leadership of the State highway department and Public Roads Administration representatives in Colorado, local subcommittees have been appointed to study and report on the road improvement program that will best meet the requirements of the county. The Delta County report describes how this particular committee approached the problems and the results of the studies.

Consideration is given to land-use and its relation to roads, schools and other community facilities, credits, marketing, public finance, land taxation, tenancy, and transportation.

PERTINENT STATISTICS PRESENTED

The following material is presented in an effort to bring out the rather simple procedure adopted by the Delta County Planning Committee with the thought of helping other counties and communities interested in land-use and its application to highway planning.

Delta County is a rather mountainous county in the western part of Colorado, covering about 1,200 square miles. The following pertinent figures are of interest:

	Acres
Total area	768, 640
National forests	190, 032
Public domain	158, 611
Irrigated land	60, 280
Dry farming land	12, 423
Fruit land	5, 891
Grazing land	223, 513
Population (1940)	16, 470
Number of farms in county (1930)	1, 898
Total assessed valuation of all land and improvements (1939)	$6, 308, 380
Average annual precipitation ...... inches	8. 3

The 1939 county tax revenue was approximately $441,000 and this sum was used for the following governmental purposes:

State	$43, 000
County	153, 000
Municipal	38, 000
General school	30, 000
Special school	177, 000

The outstanding bonded indebtedness as of 1939 was $254,000 in school bonds, $368,000 in municipal general bonds, and $37,700 in special municipal bonds. Tax collections have been below the State level and were 75.79 percent for the year 1939.

Delta County has a road system of 836.8 miles which the latest inventory shows to be divided into the classifications shown in table 1. It will be noted that over 95 per cent of the total mileage is made up of unimproved, graded and drained, and gravel surfaced roads.

The 1939 county road and bridge fund account reveals the following receipts and disbursements:

Receipts:	
Property tax	$58, 878
Specific ownership tax	2, 223
Receipts from cities	4, 844
Motor fuel tax	28, 995
Motor vehicle tax	15, 246
Motor carrier tax	2, 556
U. S. Forest Service	2, 563
Reimbursed items (labor)	460
Miscellaneous	9
Subtotal	$115, 774
Carry-over balance 12/31/38	7, 483
Total	$123, 257

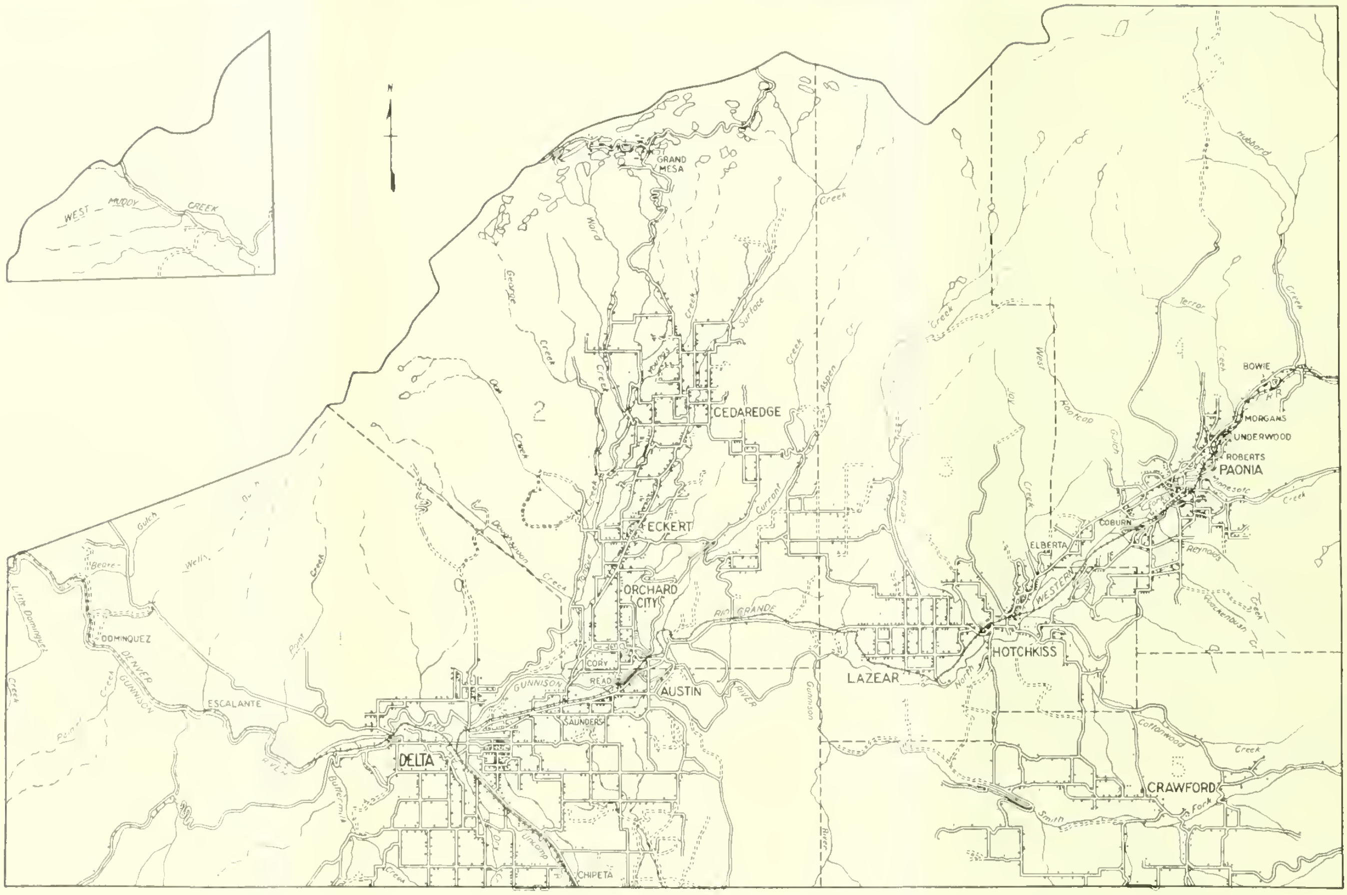

Figure 1.—Cultural Features of Delta County and Boundaries of Community Areas.

Disbursements:

Construction, maintenance, and right-of-way	$87, 955
Equipment	22, 980
Fees to county treasurer	1, 843
Transfers to other funds	6, 959
Administration	841
Subtotal	$120, 578
Carry-over balance 12/3139/	2, 679
Total	$123, 257

Table 1.—*Classification of rural roads in Delta County*

Type	Federal aid	State	County	Total
	Miles	*Miles*	*Miles*	*Miles*
Unimproved		9. 3	450. 7	460. 0
Graded and drained		41. 6	117. 3	158. 9
Gravel surfaced		76. 3	105. 0	181. 3
Low-type bituminous	24. 9	5. 2	6. 5	36. 6
Total	24. 9	132. 4	679. 5	836. 8

In undertaking a county planning program, whether it be for highways or any other purpose, coordination is essential. An alert and interested county agent is a primary requirement. It is his job to undertake the correlation of material and do the necessary contact work. Delta County is fortunate in having such an agent, and the county agricultural committee has functioned efficiently in arousing enthusiasm and maintaining an interest in the several communities.

The first step in the highway planning program for Delta County was to obtain satisfactory large-scale maps. These were obtained from the Colorado Highway Planning Survey through the Bureau of Agricultural Economics at the State College. These maps display all cultural features such as a land grid system, all cities, towns, roads, and such details as State and national forests and parks, dwellings, schools, churches, buildings, and other traffic-generating points. Upon receipt of the maps the county was divided by the county agent into five community areas as indicated in figure 1. The map was then cut along the community boundaries and the respective map sections were numbered as shown in the figure. The first county highway meeting was called at this time. Chairmen were selected to represent each community group and given the map section covering their area of the county.

MAP SHOWING TYPES OF SOIL PREPARED FIRST

The chairmen were given instructions regarding the work to be done. They were informed that each community should first prepare a map showing types of soil. It was impressed upon the farmer-rancher members that technical soils studies and terminology were not desired, but rather that the farmers and ranchers should outline the areas according to classifications understandable to their own members. Upon completion of the five community soils maps, the county agent pieced them together again to form the county map. Necessarily, some adjustment was required at the community boundaries in order to make adjoining areas coincide. Surprising as it may seem, very little correction was required. The final result was figure 2.

The question arises—Why are highway people interested in a soils map? There are several reasons. The

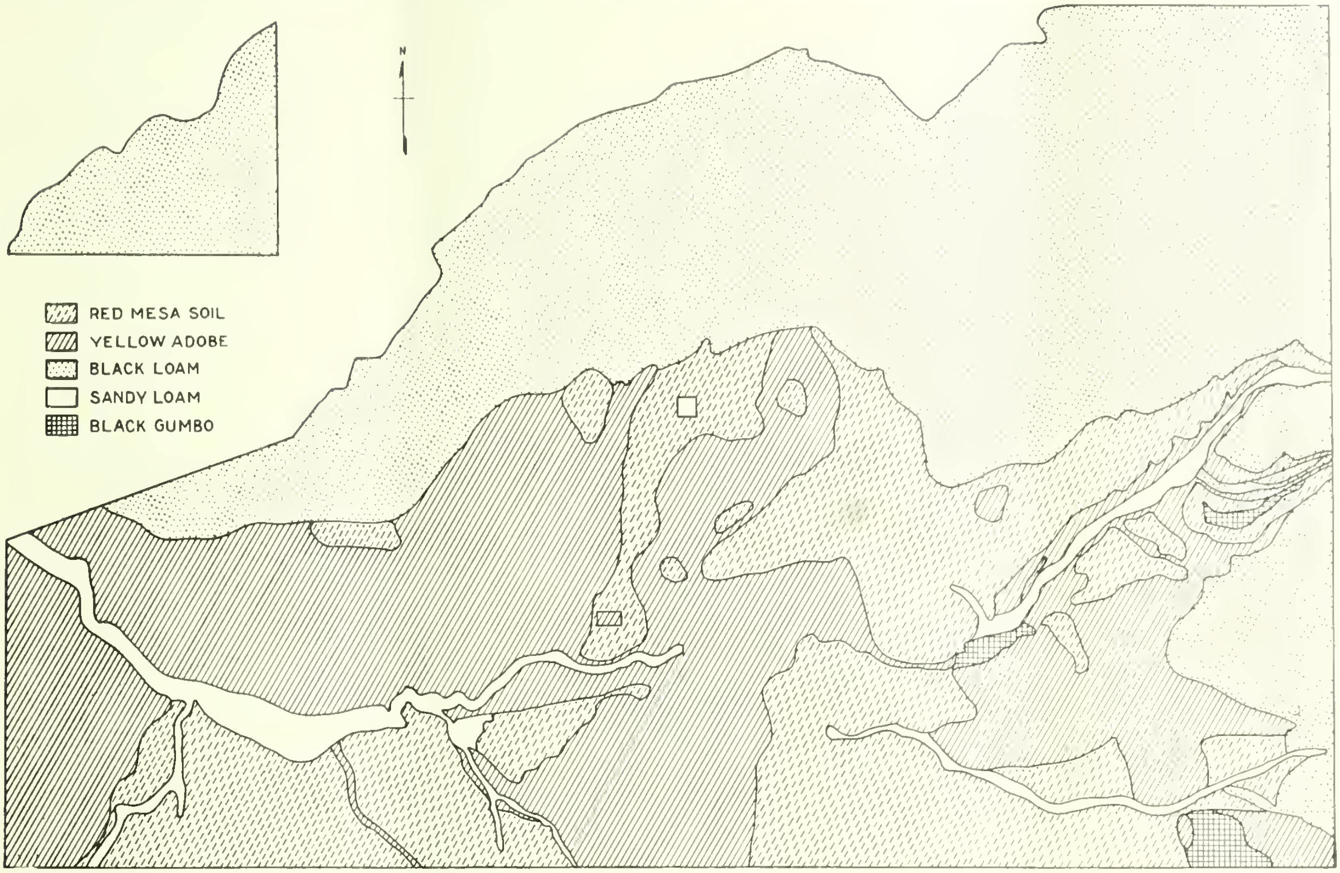

Figure 2.—Types of Soil in Delta County.

farmer and rancher know their own land; they know its capabilities and they are well qualified to state what lands are capable of production and what lands are unproductive or submarginal in character. The farmer or the rancher is far more capable of making this determination than is the highway engineer. In producing this Delta County map, the local people have shown graphically the areas of submarginal land which should not be opened to development and large-scale cultivation, thus expressing to the road builder the important fact that roadbuilding to any major degree must be discouraged in such areas. Some development may be necessary, but care must be exercised in the selection of such roads and their purpose carefully studied.

The soils map is a base for work by other agricultural agencies, for example, the Soil Conservation Service. The work being done in erosion control is well recognized, and in this map there are displayed those soils subject to erosion—those areas that are now problem areas in the county.

The next step comes under what is termed "land-use planning." The communities again were called upon to spot the actual land-use areas on their section of the map. Figure 3 shows the results of this study. Attention is directed to the grouping of existing units and how they follow the present road system. The public domain and grazing lands shown in figure 3 are not open for settlement and, except for areas within the national forests, probably represent rather unproductive lands of questionable agricultural value inasmuch as they have never been taken up in homestead. However, these areas are important to the Delta County range programs for livestock. The farming areas indicate portions of the county that require road facilities. The other sections are submarginal in character, requiring a minimum of road facilities. Normally, roads in submarginal areas are low in traffic volume and should be limited to those that are intercounty in character, those that provide small towns and communities with access to the main highways, or those confined to specific purposes, such as range management and supply.

Upon completion of the basic maps, the communities were requested to set up in their areas a road system which they felt would best serve their community needs. They were asked to ignore existing systems, to forget designated routes and present status of improvements. The thought was for the local people to discard personal prejudice and work toward an integrated road system that would provide service to the several towns and markets, to the churches, schools, social centers, and other gathering places.

The laying out of this road system by the communities immediately stimulated interest in this type of planning. Apparently everyone was concerned vitally with the highway system, especially in his community. The county agent was now called upon to do a lot of contact work. Coordination of routes, intercommunity in nature, had to be accomplished and, upon completion of the community plans, the routes had to be fitted into the complete county picture.

At this point a second county highway planning meeting was called, to which were invited representatives of each community, the county commissioners, the State highway department, the United States Forest

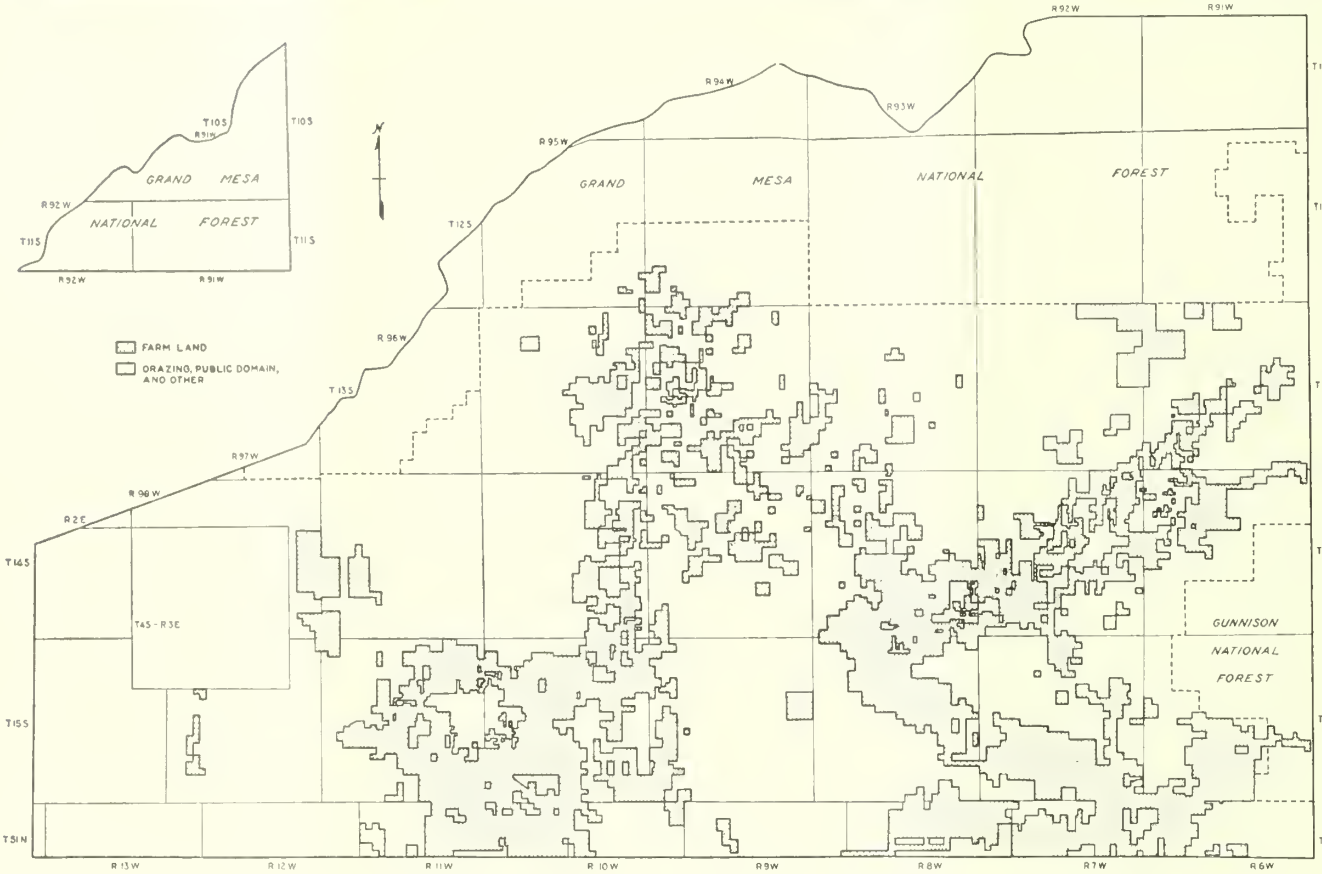

Figure 3.—Present Land Use of Delta County.

Service, and the Public Roads Administration. The county chairman explained the purpose of the meeting and requested each highway agency to indicate on the map which roads were eligible for improvement with funds from that particular agency.

ROADS OF INTEREST TO VARIOUS AGENCIES DESIGNATED

The Public Roads Administration representative indicated the Federal-aid system and those roads that were qualified Federal-aid secondary routes. The Forest Service representatives located the Forest highway system and those roads of interest to the Forest Service from the viewpoint of fire-fighting, truck trails, etc. (forest development system). In turn, the State highway department designated the State highway system. The remaining roads were the county system.

It was gratifying to note the interest displayed by the county road authorities at this meeting. Perhaps for the first time, they had been presented with factual data to substantiate the long-recognized difficult circumstances in which they find themselves in the task of providing adequate road facilities with available revenue. Factual data to aid in obtaining public support in any program that would remedy, or at least alleviate, the condition, are necessary if the problem is to be attacked successfully. The fact that such information was now available brought wholehearted support from the county commissioners.

Following this meeting, the community groups met to select a final plan which included the Federal-aid system, a series of routes designated as qualified Federal-aid secondary roads, the State highway system, and finally a county system. Figure 4 shows by appropriate symbols roads included in these several systems.

It is interesting to study roads that are requested to be abandoned and roads that are designated as a private obligation; also how the selected system fits into the soils-capability map. There are no roads into or through submarginal areas except the intercounty (Federal-aid) road running north and south, and the intercounty (Federal-aid secondary) road into the national forest. Several short stubs of county road are shown to extend into grazing and range lands for purposes of hauling stock and supplies.

Attention is directed to the mileage of "private road" serving but one or two ranches. In one instance, it was stated that several thousand dollars had been expended by the county to maintain a road serving but one ranch. Similar instances were found in other places from a study of county records.

Attention is also directed to the road marked for abandonment in the southwest part of the county. A considerable sum of county funds had been expended in gravelling this road. Subsequently the road remained unused, maintenance was neglected, and it now is to be closed. Here again the county authorities evidenced a great interest in planning. The development of an equitable and systematic county road plan acceptable to all offered an opportunity to eliminate the practice of undertaking unworthy projects such as this one.

A third county highway planning meeting was held for final adoption of the plan. This meeting was attended by all persons and groups within the county that were interested in highways. These included representatives from the chambers of commerce, civic clubs, and service organizations. The State highway advisory board member for the district was in attendance, as were State legislators representing Delta County. At this

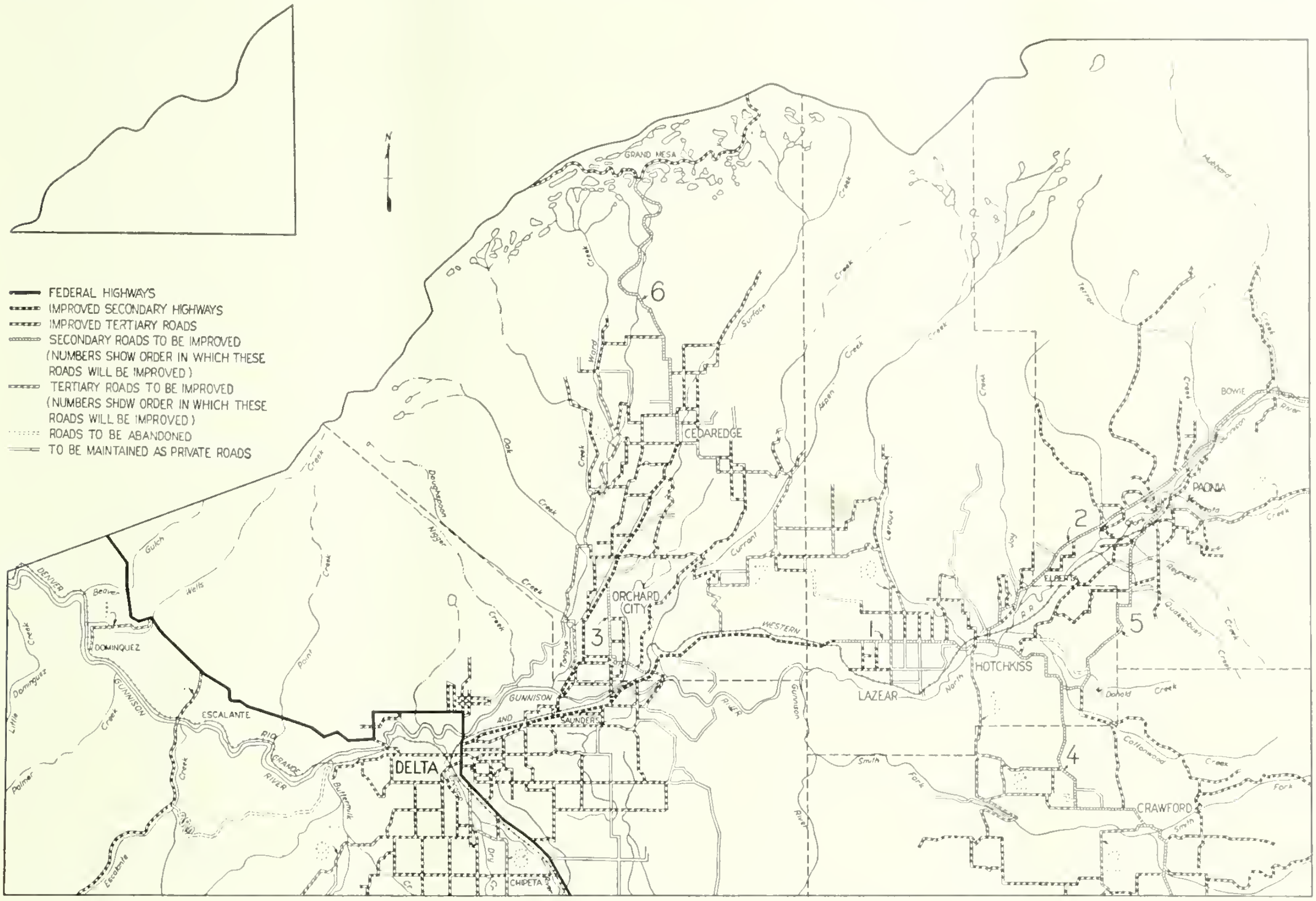
FEDERAL HIGHWAYS
IMPROVED SECONDARY HIGHWAYS
IMPROVED TERTIARY ROADS
SECONDARY ROADS TO BE IMPROVED (NUMBERS SHOW ORDER IN WHICH THESE ROADS WILL BE IMPROVED)
TERTIARY ROADS TO BE IMPROVED (NUMBERS SHOW ORDER IN WHICH THESE ROADS WILL BE IMPROVED)
ROADS TO BE ABANDONED
TO BE MAINTAINED AS PRIVATE ROADS
N
GRAND MESA
6
CEDAREDGE
PAONIA
2
ORCHARD CITY
3
1
5
HOTCHKISS
LAZEAR
4
CRAWFORD
DELTA
DOMINGUEZ
ESCALANTE
GUNNISON
RIO GRANDE RIVER
CHIPETA
BOWIE
WESTERN

Figure 4.—Proposed Reclassification of Roads into Public and Private Roads and Those Recommended for Abandonment, and Program for Road Development.

meeting all differences of opinion were aired and a system of roads finally adopted.

Each road was considered and, for each administrative system, priorities for construction were indicated on the map (fig. 4). It will be noted that the Federal-aid route (from Montrose through Delta to Grand Junction) is given no priority rating, because the local people are of the opinion that the present improvement is satisfactory and adequate for existing and anticipated traffic.

The Federal-aid secondary system as proposed is indicated by two symbols. One symbol shows portions considered satisfactory, while those portions which, in the opinion of the local people, require attention are shown by a different symbol. These latter sections are numbered from 1 to 6 in order of priority for improvement.

The county road system is also shown by two symbols; one indicating roads in satisfactory condition, the other indicating roads to be improved, again by priorities 1 to 10.

MANY MILES OF ROADS REMOVED FROM PUBLIC SUPPORT

Roads to be abandoned are shown by still another symbol. The remaining roads are considered private roads or ways of convenience, and were made the responsibility of abutting property owners or those individuals served; that is, they are no longer considered public roads. Roads in this classification are now being vacated by resolution. Thus, all road building agencies are now supplied with a long-range highway plan for the county. All interested groups and communities are united in a plan for improvement, and as funds become available, these funds can be expended to the best interests of the county as a whole.

With this plan, all of the road-building agencies will be in a position to work together on a united program of improvement which will take into consideration the needs of every county and every part of each county. The State highway advisory board member is conversant with the wishes of the county and will be able to advance Delta County's interests intelligently at the highway department budget meetings. The same holds true in programming Federal and county funds for the county.

The road system adopted as the result of these meetings and the advantages to the county are shown in table 2 and in the following recapitulation:

	Miles
Federal-aid system	24. 9
State system	81. 5
County system	433. 1
Total	539. 5
Roads abandoned	18. 6
Roads reverting to private ownership	278. 7
Total roads to be vacated	297. 3

The county has thus been relieved of responsibility for road construction and maintenance on 297.3 miles or approximately 36 percent of the road system (compare with table 1). Prior to the adoption of this plan,

TABLE 2.—*Delta County rural road mileages under land-use plan*

Condition	Federal-aid system	State system	County system	Total
	Miles	*Miles*	*Miles*	*Miles*
Unimproved	--------	--------	202.2	202.2
Graded and drained	--------	28.6	106.3	134.9
Gravel	--------	47.7	118.1	165.8
Low-type bituminous	24.9	5.2	6.5	36.6
Total	24.9	81.5	433.1	539.5

the county was attempting to service 132.4 miles of State highways (now reduced to 81.5 miles by dropping 50.9 miles) and 679.5 miles of county roads—a total of 811.9 miles—with a road and bridge fund of approximately $115,000 annually.

It would be well to point out that in Colorado the State highway laws provide that the State highway department can expend construction funds only on the State highway system, of which the Federal-aid system is a part. State highway maintenance funds can be spent only on the Federal-aid system.

During these several meetings all present were repeatedly informed and were made fully aware that completion of the program cannot be anticipated in 1 year, 5 years, or even 10 years. They are aware that situations may develop which will perhaps require modifications of the plan, yet it is a definite plan started from the grass roots. It is an intelligent plan accepted by the several road construction and maintenance agencies and consequently has a good chance to be carried to completion as the various highway funds become available.

No complete figures have as yet been obtained from the county as to the number of miles of road abandoned and petitioned for closing, as this work is still under way. It can be anticipated that many miles of private road will be closed.

A good example is in an eastern Colorado county which has progressed in this work to a point where results are now available. This county had 3,200 miles of highway prior to the time its highway planning program was set up. After proceeding along the lines already described, it was able to close 969 miles of dedicated county road without seriously affecting a single farmer or rancher. A total of 1,200 miles have been taken off the county road system by petition. These roads have reverted to private ownership, thus relieving the county of that construction and maintenance expense. The local authorities have realized their finances were limited, and with a county highway system confined to essential routes, intelligently and efficiently selected, these limited funds can be expended to provide an adequate well-improved road system which will meet the entire county road requirements.

The Delta County Planning Committee, realizing the vital need for wise and efficient administration of road funds, is now educating its people to plan for the future, and its excellent work in highway planning shows intelligent application of simple planning principles applied by unselfish interests.

Delta County is going further, as have several other counties in the State. A systematic review is being made of all school-bus and mail routes. Working with the board of education, school busses are being rerouted for more efficient service. One county in eastern Colorado has accomplished this, and reports a saving in contracts for student haul in the last six months of 1940, which enabled it to purchase a new school bus to replace an obsolete model. Likewise, working with postal authorities, mail routes have been changed to provide more economical service and prompter mail deliveries.

Admittedly, selfish interests must be overcome. Were the State or the Federal governments to attempt to reroute school busses or mail delivery, the local people would no doubt protest. Yet these same local people are now accomplishing this economy, requiring their children, in some instances, to walk half a mile to the bus line, or perhaps themselves walking some distance to their rural free delivery mail box. The school bus and the mail carrier no longer come up to their very front door.

OTHER PLANNING WORK BEING DONE

Other outgrowths of the local land-use meetings have been both surprising and encouraging. The subject of roadside development is an example. The State and Federal highway organizations, of course, are interested in this work. Limited amounts of funds have been expended during the past few years on this type of work. The landscaped areas adjacent to the highways in various parts of the State have, however, been all too few. Lack of moisture in most parts of Colorado precludes extensive landscape development because of prohibitive maintenance costs. However, there are many quick-growing shrubs and trees that are indigenous to the State and can be used for roadside development and parking areas.

In many communities, the women members of the county and community land-use planning groups have become interested in the creation of such areas. The State highway department has a landscape engineer who will cooperate with the county groups to create such areas and assist in solving problems. The State Forester has available thousands of trees suitable to the locality and will gladly supply them at a nominal price—in some instances as low as 15 cents per tree. The women are planning to develop some of these areas this year. The local property owners are contributing small strips of land to permit adequate turn-out facilities and the highway department of the State or county is contributing the necessary grading and approaches. The local groups are completing the planting and arranging for maintenance as a matter of civic pride. Larger projects of this kind should be undertaken at logical places, and as State and Federal funds become available these projects can be set up by the State.

It would seem proper to review briefly other commendable planning work done in Delta County. Certain problems have arisen regarding soil erosion. Here again the communities have mapped these respective areas, spotting critical erosion areas (see fig. 5) where the local people feel they have insufficient means to control the damage and prevent further destruction of the soil. This map also displays problem areas where low soil fertility is causing serious difficulty. The Soil Conservation Service and State College are now engaged in remedial measures to assist the local people in solving this problem.

Figure 6 displays water shortage problem areas. Here again the affected farmers and ranchers are working with the proper action agencies and the advice and experience of government technicians are helping to solve the problem.

Figure 7 displays the excessive seepage areas and areas where there is a shortage of water for domestic

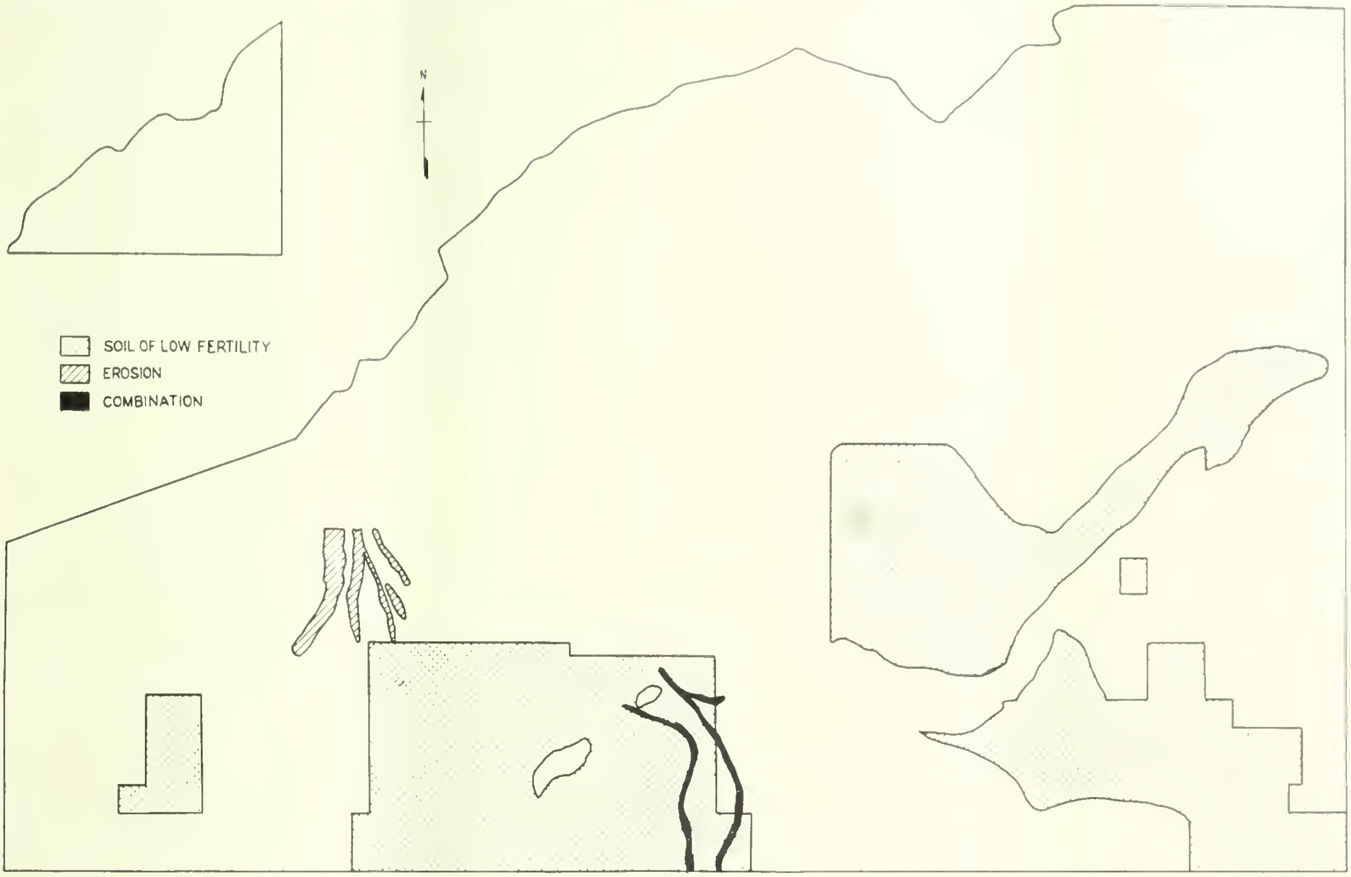

FIGURE 5.—AREAS HAVING LOW FERTILITY AND EROSION PROBLEMS.

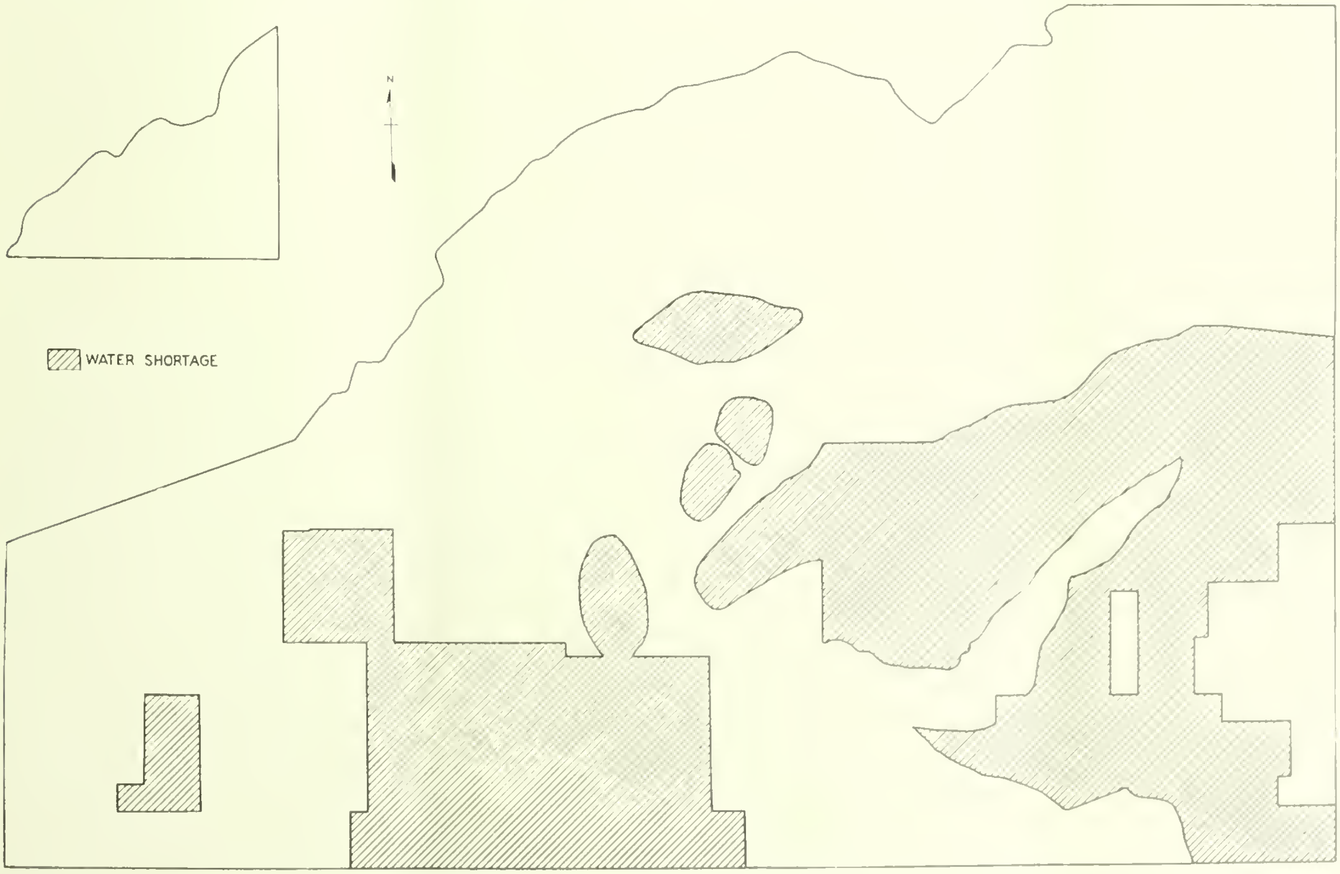

FIGURE 6.—WATER SHORTAGE AREAS.

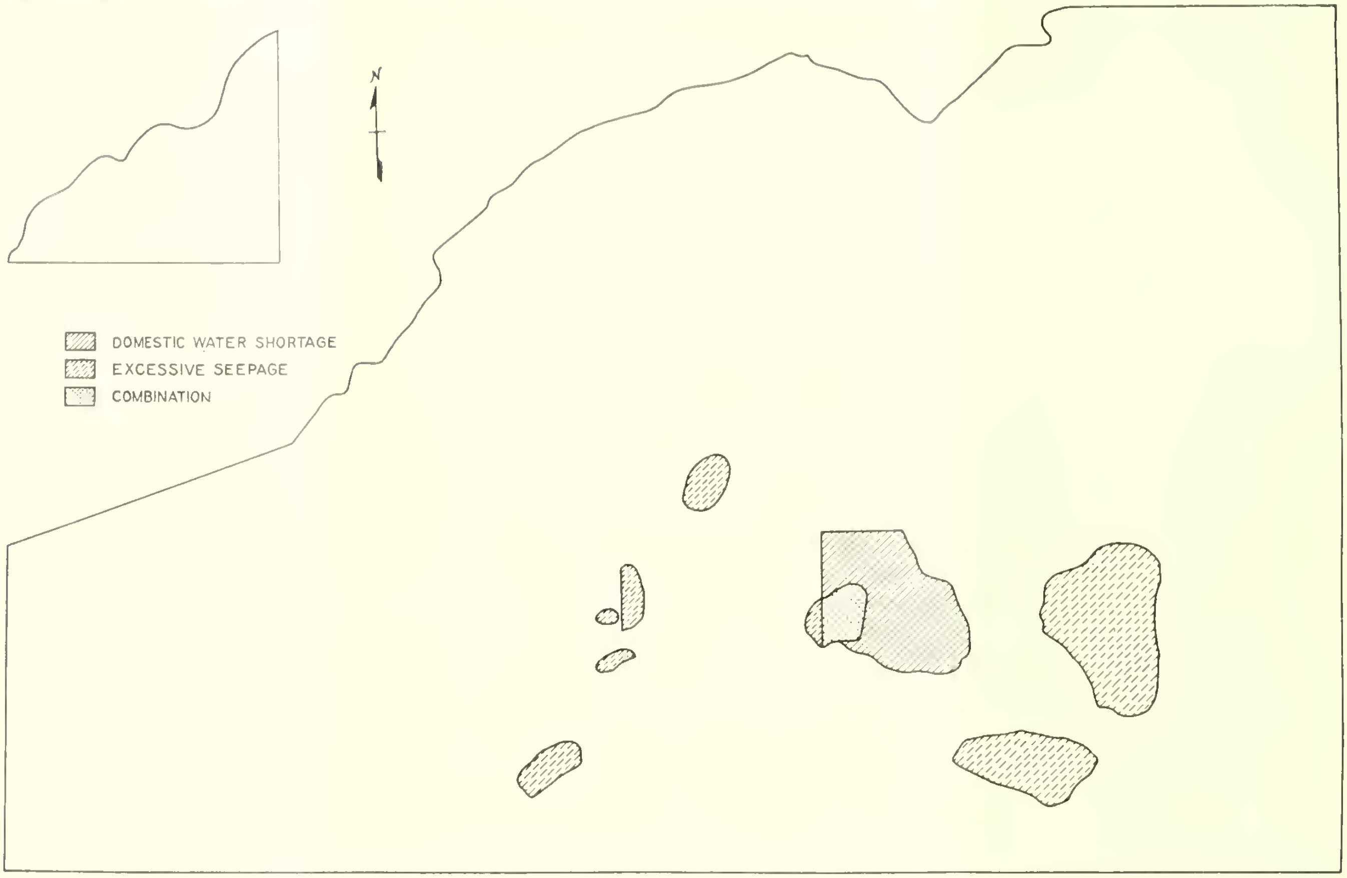

Figure 7.—Areas Where There Is a Shortage of Domestic Water, and Areas of Excessive Seepage.

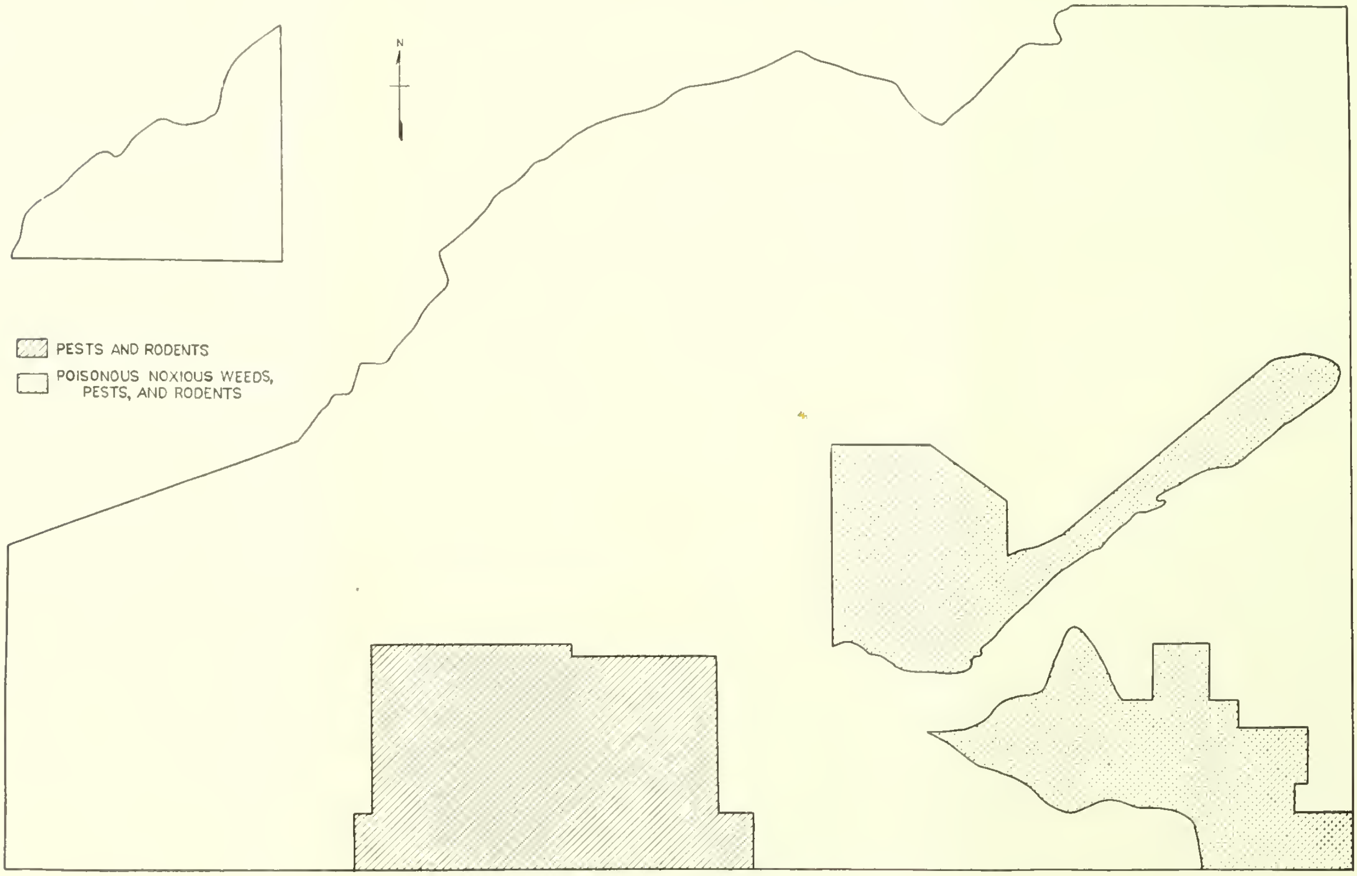

Figure 8.—Pest and Weed Problem Areas.

use. The several State and Federal agencies are contributing time and funds to remedy this situation.

Figure 8 shows the pest and weed problem areas in the county and here progress is being made toward solving the problem through asistance given by the Biological Survey and other interested agencies.

It can be seen that all of the work accomplished by Delta County in land-use planning requires time and coordinated effort. Those counties that have undertaken this task are to be complimented. At this time many of the Colorado counties are in some stage of the work, and by the end of 1941 over a third of the 63 counties will have undertaken plans for a long-range program similar to that of Delta County.

In conclusion, it would be well to stress the vital need for coordination of effort. The State Agricultural Planning Committee is encouraging constructive planning for the coordination of activities within the State, and the country groups should place themselves behind this movement and stress the need for continuous efforts in highway planning. Expenditures for highways into submarginal areas should be discouraged. Planning for highways must insure efficient operation of a system of highways that meets the best interests of the individual county, the State, and the Nation.

(*Continued from p. 4.*)

In dealing with runoff problems it may be helpful to review briefly the physical phenomena which take place. To begin with, rain never falls at a uniform rate for long intervals. Rain falling at low intensities may be completely soaked up in the soil, this infiltration taking place rapidly at first and then at a minimum sustained rate regardless of how long rain may continue to fall. This infiltration rate depends on the size and compaction of soil particles, presence of organic material in the soil and on the surface (in the form of vegetation or mulches), and other factors. Rain falling on bare soil beats the soil into a muddy suspension which tends to clog the surface pores, thus decreasing infiltration. A mulch protects the soil from this action.

During the time that the rate of rainfall exceeds the rate of infiltration, runoff takes place. The small depressions fill up first and overflow until finally a sheet of water of appreciable depth is creeping slowly over the surface. From that point on the rapidity with which the water concentrates depends upon the number and kind of collecting channels.

Since a smoothly paved surface permits little or no infiltration, and offers little resistance to flow, the volume of water detained on the surface at any time is very small and the peak rate of runoff is very nearly equal to the peak rate of rainfall. On the other hand, a gently sloping bluegrass pasture has a high rate of infiltration, affords much resistance to surface flow, and can store up a large volume of water with the result that the peak rate of runoff may be only one-tenth the peak rate of rainfall. The stored-up water will flow out gradually over a long period of time, perhaps an hour or two after rainfall has ceased.

The frequency with which any peak rate of rainfall occurs in a given region can be determined from intensity–frequency data such as are contained in United States Department of Agriculture Miscellaneous Publication No. 204. A given intensity of rainfall will seldom produce the same peak rate of runoff in successive occurrences because the characteristics of the drainage area affecting runoff will vary within a wide range. Consequently a 2- or 5-year rainfall frequency may be safely used as equivalent to a longer runoff frequency. In attempting to evaluate the damage that may be done by runoff rates exceeding that for which a channel has been designed, it may be noted that on a given drainage area, the higher the peak rate the shorter the duration of that rate. The principal effect is to superimpose a sharper and higher peak on the runoff curve. The greater the opportunity for temporary surface storage, the less this peak will exceed the normal peak.

METHOD OF DESIGNING ROADSIDE CHANNELS SUGGESTED

Due to the uncertainties necessarily involved in estimating runoff from small drainage areas the simple calculations should not be carried out to more than one or two significant figures. Drainage area divides may be spotted by field parties taking preliminary cross sections, and plotted on a drainage area map preferably drawn to a fairly large scale. The estimation of runoff coefficients or the selection of runoff curves should not be done in the office by a draftsman, but should be done in the field by an engineer familiar with factors influencing runoff and able to visualize conditions likely to exist after construction is completed.

After peak rates of runoff have been estimated, a diagram of the type shown in figure 1 may be used to check the velocity developed in, and capacity of, the standard channel section ordinarily used. In the great majority of cases the capacity of the standard section will be found adequate. In a few cases the computed velocity will indicate the need for sodding. Infrequently, critical conditions will necessitate special study to develop economical designs.

The purpose of introducing this method of analysis is not to disrupt procedures of preparing plans developed through long years of experience but to add a simple check on channel design which will reveal, during the planning stage, the points at which extra care should be taken to avoid future difficulty and expense in maintaining the completed highway.

Detailed discussion of the construction and maintenance of vegetated channels is beyond the scope of this paper, but a few points should be noted. The kind of grass selected for seeding or sodding a channel should preferably be a short-bladed variety with a deep root system forming a dense turf and not a bunch grass around which the water will wash, or a stiff-stemmed grass which will not bend flat under the pressure of the current. The efficacy of sod for controlling erosion results largely from the "shingling" or "thatching" action which protects the soil from the rapidly flowing water in the stream. A favorable circumstance is the fact that in most regions the highest intensity rains, which cause the most erosion damage, normally occur during the summer months and not when the grasses are dormant. Experiments with bermuda grass, however, indicate that its resistance to erosion is practically as good when the grass is dormant as when it is green.

When seeding can be done at the right season of the year to establish a reasonably good turf in a short period of time, channels from very small drainage areas may be seeded instead of sodded, taking the chance that a damaging rain will not occur during that period. Any necessary reseeding will probably still involve less total cost than an initial installation of sod. Mulching of the seeded areas, now rather common practice, greatly decreases the chances of damage by intense rains. Local circumstances and individual judgment will determine the extent to which chances may be taken on seeding in drainage channels.

In some regions grass is normally established by sprigging or planting stolons and roots, relying on the natural spreading habit of the particular species to establish complete cover in a short period of time. In such regions the remarks about seeding will apply with equal significance to these other inexpensive methods of establishing grass.

A compromise solution is to sod only the bottom of the channel, (assuming the bottom to be rounded) on the chance that a rate of runoff sufficient to rise above the sodded area will not occur until grass has become established by other, cheaper methods on the rest of the channel. To play safe, however, sod should be placed to an elevation slightly above the depth of the peak discharge for which the channel is designed (this procedure is recommended where sod is plentiful and relatively inexpensive).

Vegetated channels can function satisfactorily only if they are adequately maintained. Channels should be mowed regularly to avoid excessive restriction to flow and to keep down weeds. Bare spots should be repaired by sodding immediately upon discovery, as small breaks in the sod enlarge rapidly when subjected to heavy flows. Silting of the channel is a troublesome problem that can be permanently solved only by tracing the silt back to its source and eliminating erosion at that point. Since silt is deposited only when the carrying capacity of the stream is reduced by checking the velocity, channels should be designed, insofar as practical, so that the gradient is always increased, and never flattened, in the direction of flow.

HIGHWAY COST AND TAXATION STUDY PUBLISHED

"Analysis of Highway Costs and Highway Taxation With Application to Story County, Iowa," has recently been published as Iowa Engineering Experiment Station Bulletin No. 152.

This bulletin, by Dr. E. D. Allen, research economist of the Iowa Engineering Experiment Station, is a study of highway costs and taxation. The several general theories of highway finance are discussed at length, and the four current interpretations of highway costs are examined. Finally, highway costs and finances in Story County during the period 1913–38 are analyzed.

Single copies of the 128-page bulletin may be obtained without charge from the Iowa Engineering Experiment Station, Iowa State College, Ames, Iowa.

ESTIMATED MOTOR-FUEL USAGE AND MOTOR-VEHICLE REGISTRATION IN 1941

State	Motor-fuel usage				Motor-vehicle registration	
	Total, 1941	Total use in 1940	Increase, 1941 over 1940		Total, 1941	Increase over 1940
			Amount	Percentage		
	1,000 gallons	*1,000 gallons*	*1,000 gallons*			
Alabama	315,155	263,861	51,294	19.44	383,500	43,647
Arizona	127,484	111,516	15,968	14.32	145,150	6,747
Arkansas	219,707	189,811	29,896	15.75	289,550	32,373
California	2,181,979	1,948,880	233,099	11.96	2,995,000	221,341
Colorado	261,762	246,918	14,844	6.01	367,400	15,290
Connecticut	425,258	380,001	45,257	11.91	551,300	57,633
Delaware	67,554	62,800	4,754	7.57	80,200	8,437
Florida	478,670	405,415	73,255	18.07	557,850	62,640
Georgia	450,973	397,987	52,986	13.31	561,950	59,347
Idaho	117,384	107,644	9,740	9.05	171,000	7,660
Illinois	1,704,892	1,547,958	156,934	10.14	2,060,400	134,586
Indiana	800,612	700,360	100,252	14.31	1,070,400	76,394
Iowa	617,993	578,061	39,932	6.91	825,100	31,131
Kansas	527,516	493,514	34,002	6.89	617,350	34,682
Kentucky	350,019	301,902	48,117	15.94	499,350	35,623
Louisiana	315,743	269,916	45,827	16.98	438,800	73,371
Maine	172,246	157,361	14,885	9.46	224,250	18,354
Maryland	358,879	312,745	46,134	14.75	496,500	51,968
Massachusetts	807,152	744,903	62,249	8.36	965,300	61,457
Michigan	1,361,682	1,223,811	137,871	11.27	1,598,350	45,789
Minnesota	626,428	576,401	50,027	8.68	911,600	40,249
Mississippi	244,545	210,349	34,196	16.26	303,750	44,737
Missouri	770,946	703,256	67,690	9.63	984,450	62,650
Montana	147,077	134,974	12,103	8.97	198,550	7,518
Nebraska	252,432	236,852	15,580	6.58	417,300	5,184
Nevada	48,526	42,487	6,039	14.21	48,200	4,092
New Hampshire	102,972	95,827	7,145	7.46	141,800	6,416
New Jersey	1,007,621	923,490	84,131	9.11	1,173,150	86,184
New Mexico	118,845	108,834	10,011	9.20	130,500	5.444
New York	2,061,581	1,970,354	91,227	4.63	2,873,450	130,436
North Carolina	541,106	459,025	82,081	17.88	653,500	61,554
North Dakota	155,839	146,215	9,624	6.58	192,950	10,663
Ohio	1,584,236	1,430,533	153,703	10.74	1,972,250	53,321
Oklahoma	470,054	440,591	29,463	6.69	592,500	17,781
Oregon	302,028	264,654	37,374	14.12	430,450	36,872
Pennsylvania	1,713,174	1,581,975	131,199	8.29	2,307,600	161,751
Rhode Island	146,915	133,758	13,157	9.84	200,750	13,241
South Carolina	279,850	234,307	45,543	19.44	391,750	54,978
South Dakota	153,563	145,480	8,083	5.56	203,400	7,733
Tennessee	380,036	323,966	56,070	17.31	512,550	64,567
Texas	1,575,064	1,407,394	167,670	11.91	1,816,900	122,787
Utah	116,158	105,635	10,523	9.96	150,600	10,679
Vermont	75,576	70,806	4,770	6.74	97,650	4,005
Virginia	504,639	424,430	80,209	18.90	571,950	73,112
Washington	430,264	384,359	45,905	11.94	616,350	53,855
West Virginia	246,928	222,210	24,718	11.12	336,700	34,142
Wisconsin	619,584	575,119	44,465	7.73	957,900	56,985
Wyoming	80,242	70,753	9,489	13.41	91,150	5,493
Dist. of Columbia	194,503	169,127	25,376	15.00	176,750	14,836
Total	26,613,392	24,038,525	2,574,867	10.71	34,355,100	2,329,735

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF FEBRUARY 28, 1942

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$6,456,558	$3,207,060	228.2	$4,012,834	$1,991,205	115.2	$415,400	$206,050	0.3	$2,765,761
Arizona	1,535,097	1,073,116	65.6	1,286,108	916,511	43.1	384,302	254,599	8.4	2,050,364
Arkansas	3,633,486	1,666,991	57.4	1,173,174	585,470	60.1	66,680	33,340	.1	1,786,711
California	8,073,437	4,334,333	138.0	5,895,451	3,504,578	67.5	472,203	307,723	14.3	4,770,191
Colorado	2,323,253	1,290,718	154.6	3,508,479	2,067,537	155.6	919,347	512,943	35.4	2,516,033
Connecticut	1,419,554	696,181	17.1	1,715,156	821,787	18.4	481,485	239,471	9.4	1,228,870
Delaware	610,532	299,944	17.1	441,010	216,559	9.0	268,040	134,020	8.4	1,496,599
Florida	1,167,179	582,930	65.3	3,254,154	1,820,890	50.7	512,242	293,621	4.6	3,080,497
Georgia	2,994,295	1,495,854	111.4	7,235,800	3,627,650	272.4	3,078,525	1,539,263	131.7	7,032,315
Idaho	1,791,533	1,172,887	93.7	1,302,194	929,085	63.1	58,224	36,000	.1	1,981,206
Illinois	4,119,631	2,038,666	89.7	7,033,216	3,516,036	121.7	1,361,200	680,600	6.2	7,131,471
Indiana	4,528,531	2,096,487	74.4	5,302,939	2,572,704	75.6	2,191,200	1,095,600	32.7	2,869,329
Iowa	3,928,203	1,887,687	170.6	4,260,316	1,752,582	125.6	581,933	97,150	17.7	2,533,496
Kansas	4,625,910	2,354,279	260.1	5,260,573	2,631,145	239.1	1,914,950	1,052,838	65.3	4,807,666
Kentucky	3,756,759	1,853,568	138.6	6,465,362	3,091,041	127.2	2,004,624	935,397	15.1	2,012,929
Louisiana	1,031,219	521,393	24.2	1,820,379	1,140,826	37.8	2,553,917	1,251,767	56.3	4,304,975
Maine	953,093	469,404	26.8	2,070,392	1,061,696	27.8	78,610	39,305	.1	1,125,422
Maryland	2,765,267	1,381,529	29.6	3,446,909	1,466,154	15.0	35,000	17,500		1,587,207
Massachusetts	2,348,774	1,177,018	17.3	2,228,103	1,147,862	14.9	1,276,508	660,427	9.1	3,829,121
Michigan	8,422,298	4,109,603	174.2	2,919,748	1,547,374	50.6	886,300	452,325	9.3	3,847,750
Minnesota	4,484,865	2,207,751	378.0	9,504,473	4,708,665	388.4	368,018	184,009	20.1	3,497,493
Mississippi	5,332,567	2,602,918	283.3	3,723,624	1,864,462	218.1	42,100	24,000		2,306,444
Missouri	5,049,795	2,543,652	160.1	10,409,439	5,704,391	193.2	1,974,782	774,309	24.9	4,354,096
Montana	2,181,671	1,233,971	117.1	3,908,728	2,283,989	169.0	518,453	299,012	60.1	4,470,051
Nebraska	2,893,320	1,419,807	279.3	5,261,073	2,646,991	478.2	695,984	347,992	37.8	4,181,574
Nevada	2,235,406	1,914,788	110.6	742,339	643,031	22.0	274,686	238,701	3.4	1,417,147
New Hampshire	339,179	177,926	6.0	1,710,877	1,021,975	20.2				853,558
New Jersey	2,999,330	1,467,761	26.6	2,988,902	1,494,371	16.2				3,041,541
New Mexico	1,632,596	1,016,478	112.8	1,098,866	778,893	63.2				2,827,672
New York	9,605,419	4,820,744	126.7	7,910,755	5,051,640	80.6	1,083,700	633,400	12.8	5,184,248
North Carolina	3,947,309	2,083,865	162.0	2,943,504	1,597,189	123.7	991,030	473,585	23.4	3,500,407
North Dakota	3,218,183	1,828,431	287.6	2,566,572	1,368,670	203.7	2,448,925	1,236,615	205.5	4,187,226
Ohio	9,513,316	4,750,920	91.7	11,309,572	5,413,691	76.8	5,465,320	2,167,658	29.7	4,458,134
Oklahoma	2,423,158	1,223,971	112.7	2,450,022	1,292,789	46.7	1,873,280	979,830	66.1	6,271,954
Oregon	2,684,938	1,585,246	72.9	3,299,713	1,718,698	71.5	403,168	294,330	30.9	1,616,095
Pennsylvania	10,068,733	4,993,380	98.5	8,522,848	4,178,504	64.5	3,094,286	1,967,432	24.9	4,453,507
Rhode Island	1,196,941	596,510	10.0	644,776	322,368	4.8	644,448	322,224	2.0	940,591
South Carolina	2,027,604	932,297	87.3	3,945,029	2,377,067	90.0	959,119	547,497	26.7	1,594,094
South Dakota	2,218,282	1,294,086	266.9	4,812,831	3,179,872	544.1	644,800	381,560	69.6	2,868,727
Tennessee	3,884,513	2,046,955	105.3	4,895,864	2,907,905	92.6	500,230	297,672	15.2	3,539,138
Texas	9,975,401	4,873,055	505.9	12,578,570	5,938,706	415.2	2,298,400	936,410	79.2	9,509,149
Utah	1,206,046	907,446	53.8	1,761,015	1,325,602	41.3	87,256	65,634	5.3	1,286,573
Vermont	831,235	429,145	28.8	1,192,668	694,327	20.7	36,906	18,453	.3	434,096
Virginia	3,934,015	1,839,199	74.7	3,460,798	1,687,536	57.3	35,490	17,745	.4	2,941,214
Washington	1,612,928	838,027	25.7	2,671,742	1,429,754	31.4	43,686	23,400	1.0	2,246,300
West Virginia	2,778,324	1,380,071	51.4	2,482,738	1,231,611	29.3	338,900	166,750	1.8	2,199,155
Wisconsin	2,195,334	1,077,854	92.7	5,618,496	2,633,643	163.9	1,262,339	454,400	44.6	4,798,393
Wyoming	1,446,636	912,464	148.3	1,773,023	1,317,527	123.5	34,714	22,321	.1	1,365,245
District of Columbia	578,415	285,730	3.3	721,662	396,682	1.1	600,000	382,500	.2	314,642
Hawaii	133,296	66,648	2.8	1,085,460	720,206	10.9	180,504	161,032	3.5	1,844,846
Puerto Rico	470,767	232,845	3.8	2,150,735	1,062,205	13.1	190,540	133,685	.2	724,111
TOTALS	169,589,138	87,293,589	5,840.5	198,779,061	105,461,652	5,570.6	46,631,754	23,392,095	1,214.2	155,985,334

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF FEBRUARY 28, 1942

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$1,392,972	$695,108	62.5	$517,831	$281,500	25.2	$206,101	$96,530	6.0	$585,420
Arizona	245,110	174,247	14.1	15,183	13,767		126,598	61,439	6.8	548,054
Arkansas	610,518	233,221	33.1	348,210	174,039	26.3	135,644	67,822	.2	339,836
California	940,338	542,934	18.3	806,159	630,393	7.6				1,067,018
Colorado	157,106	82,891	20.7	129,755	72,650	2.1	199,327	61,919	6.3	505,877
Connecticut	298,035	136,331	6.1	266,247	115,938	4.8				199,604
Delaware	81,076	38,116		224,923	111,986	12.3	102,873	37,618	3.9	246,037
Florida	498,886	249,443	4.7	666,633	338,767	7.1	161,570	80,680	5.9	402,653
Georgia	545,666	257,833	43.4	1,143,478	664,089	71.6	461,992	230,996	47.0	1,125,176
Idaho	293,347	172,516	26.2	187,185	118,985	7.7	78,125	48,303	5.9	299,311
Illinois	1,073,034	518,949	59.4	1,147,660	573,830	54.5	96,500	48,250	10.6	903,371
Indiana	600,171	295,687	39.8	1,128,455	531,071	46.7	189,600	94,800	6.4	936,690
Iowa	597,238	281,841	148.3	398,264	170,008	64.8	346,551	161,825	60.5	588,518
Kansas	773,140	387,620	98.3	1,650,915	830,957	98.6	460,565	230,282	36.9	1,039,033
Kentucky	1,161,015	321,329	83.2	1,267,862	388,683	37.2	74,213	24,400	5.6	353,316
Louisiana	571,169	233,519	20.6	7,700	3,850		289,362	138,761	21.5	698,808
Maine	77,540	38,770	3.6	235,218	117,609	10.5	16,850	2,714	.4	161,104
Maryland	473,000	236,500	19.5	333,376	166,513	3.1				345,564
Massachusetts	179,789	93,569	4.1	663,234	352,683	10.1				572,305
Michigan	1,218,371	600,611	76.3	780,498	390,249	24.8	315,969	157,985	11.2	641,630
Minnesota	1,598,379	799,373	217.1	973,917	484,197	86.9	300,776	149,988	30.1	605,414
Mississippi	829,715	414,858	46.5	1,199,746	533,738	47.6	274,300	137,150	11.5	436,055
Missouri	434,916	215,032	52.9	868,872	416,843	89.5	234,254	87,198	36.9	1,053,224
Montana	377,420	214,407	58.5	292,724	170,722	31.0	13,569	7,715	4.5	889,999
Nebraska	359,864	179,301	49.4	487,813	248,520	64.7	31,940	15,970	9.2	684,152
Nevada	224,109	185,777	24.1	92,413	60,816	4.6	99,973	79,290	4.5	235,306
New Hampshire	152,914	75,436	4.7	241,629	119,915	3.6				152,254
New Jersey	497,637	244,471	9.6	412,615	230,410	12.1	51,500	25,750	1.8	570,561
New Mexico	566,935	357,245	47.7	189,504	122,533	15.2				296,856
New York	959,667	469,941	28.8	894,946	488,111	14.3	485,666	242,833	1.5	1,095,984
North Carolina	339,731	177,102	34.9	529,507	285,848	36.3	69,820	20,000	5.0	681,541
North Dakota	29,802	15,664	2.4	7,382	7,382		808,050	793,860	42.7	755,172
Ohio	1,777,334	886,683	57.6	883,130	489,535	14.5	71,000	35,500	2.0	1,388,480
Oklahoma	363,550	191,497	21.6	64,572	34,093	9.9	903,706	477,157	64.1	1,083,038
Oregon	463,659	243,290	41.8	459,611	217,324	28.5	30,482	18,000	1.3	405,495
Pennsylvania	1,727,496	863,074	32.0	780,163	379,689	16.0	73,583	36,794	1.8	777,528
Rhode Island	220,879	111,427	2.6	22,194	18,197					130,911
South Carolina	787,356	307,866	54.6	221,700	79,945	.5				397,720
South Dakota	33,337	18,686	15.2	7,778	7,778		1,143,430	1,047,600	114.5	761,862
Tennessee	468,207	232,400	14.1	1,337,954	668,977	46.0	158,042	79,021	4.5	770,269
Texas	1,314,680	634,272	132.4	711,479	347,847	62.8	43,500	21,700	4.8	2,253,336
Utah	186,949	123,241	17.0	153,666	101,709	6.8				337,043
Vermont	40,708	18,109	1.2	180,204	85,595	7.8	42,311	31,733	1.1	53,580
Virginia	370,460	174,585	15.8	346,346	154,866	4.3	59,650	29,825	4.5	676,248
Washington	279,641	160,072	17.8	454,148	211,574	13.7				413,564
West Virginia	400,342	202,347	19.8	328,314	163,544	6.4				501,897
Wisconsin	936,706	467,669	42.7	1,406,919	646,743	46.0	64,461	31,320	.8	595,789
Wyoming	357,528	158,064	18.8	508,423	218,112	34.3				220,633
District of Columbia	76,620	37,756	.9	2,558	1,279					163,600
Hawaii				2,375	2,375					334,875
Puerto Rico	170,027	77,541	7.5	73,642	38,810	3.1				217,968
TOTALS	28,134,089	13,848,221	1,872.2	26,055,030	13,084,599	1,221.4	8,221,858	4,912,728	582.2	30,500,579

U. S. GOVERNMENT PRINTING OFFICE: 1942

PUBLICATIONS of the PUBLIC ROADS ADMINISTRATION

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Agency and as the Agency does not sell publications, please send no remittance to the Federal Works Agency.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1932. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1938. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1939. 10 cents.

Work of the Public Roads Administration, 1940.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . . The Results of Physical Tests of Road-Building Rock. 25 cents.

No. 191MP. . Roadside Improvement. 10 cents.

No. 272MP. . Construction of Private Driveways. 10 cents.

No. 279MP. . Bibliography on Highway Lighting. 5 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. 60 cents.

Highways of History. 25 cents.

Specifications for Construction of Roads and Bridges in National Forests and National Parks. 1 dollar.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.

No. 265T. . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Public Roads Administration upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP. . Bibliography on Highway Safety.

House Document No. 272 . . . Toll Roads and Free Roads.

Indexes to PUBLIC ROADS, volumes 6–8 and 10–21, inclusive.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF FEBRUARY 28, 1942

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	NUMBER			Estimated Total Cost	Federal Aid	NUMBER			Estimated Total Cost	Federal Aid	NUMBER			
			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$151,956	$151,436	2	4	2	$383,625	$381,603	5	2	2	$52,335	$52,335	2		5	$1,129,271
Arizona	184,378	184,361	1	1		125,274	116,583		1		13,255	13,255			2	234,779
Arkansas	464,676	463,371	5		5	166,996	165,286	1		1	22,858	22,858			7	683,334
California	833,918	644,346	2	1	4	870,516	868,399	8			12,450	12,450			4	2,319,005
Colorado	5,685	5,646				664,333	664,333	7			21,042	21,042			10	730,378
Connecticut	166,222	165,415	2			61,712	60,676		1		231,374	222,740		1		525,593
Delaware	89,725	89,725	1		1	191,599	189,867	1	1		508,406	321,735	2			179,564
Florida	120,961	120,961			20	733,446	731,266	8		6	187,620	187,560			14	875,288
Georgia	660,009	660,009	7	2	5	843,014	843,014	5	6	10	943,446	943,446	3	4	14	1,515,675
Idaho	25,827	21,580			2	322,273	313,602	4	1		6,212	6,212			3	427,671
Illinois	687,682	565,714	2		82	1,667,042	1,572,807	8	1	9	414,244	395,294	1	1	35	2,561,553
Indiana	606,900	594,413	6		20	464,035	458,326	2	1	21	96,164	96,164			23	1,174,656
Iowa	353,794	342,320	3	1	28	1,444,738	1,192,530	10	2	15	176,038	174,725			43	663,450
Kansas	67,313	66,808	2		6	679,768	679,768	8		1	212,835	208,304	4		10	1,316,576
Kentucky	1,111,122	1,108,759	9		1	441,711	441,711	4	1							410,133
Louisiana	6,965	6,965				586,220	586,220	8			481,835	480,667	4		4	923,884
Maine						363,086	363,086	2			8,680	8,680			3	288,623
Maryland	488,250	456,457	2	2	10	879,908	736,111	4		5	37,325	37,325			7	399,876
Massachusetts	346,270	335,829	1		1	774,431	773,559	5	2		763,830	763,830	2	1		1,267,765
Michigan	1,314,447	1,307,560	3	4	19	253,416	253,416	1		5	333,180	303,019	1		11	1,325,454
Minnesota	532,378	532,064	5	3	1	1,074,944	1,074,944	5	5	4	59,688	59,688		1	4	1,194,726
Mississippi	235,874	235,874	2	1	4	861,110	861,110	10		6	25,808	25,808			2	645,010
Missouri	120,702	120,702		2		1,922,921	1,467,501	6	2		464,353	464,353	2	2		1,489,049
Montana	141,549	141,549	2		2	99,778	99,778	1								733,051
Nebraska	200,886	200,495	2	1	22	1,142,413	1,142,413	21		1	13,020	13,020			3	451,612
Nevada	119,580	119,580	2			59,085	59,085	2		1	28,124	28,124			9	192,599
New Hampshire	261,080	260,783	4	1	2	100,861	100,536	2	1	3						313,952
New Jersey	845,837	844,536	4	1	1	629,879	504,329	3	1	1	354,085	295,560	1	2		939,833
New Mexico						68,342	68,342	1			259,103	252,068	3		1	521,824
New York	2,422,733	2,364,650	2	13		1,962,196	1,921,057	3	8		502,645	464,285		1		3,747,433
North Carolina	518,515	518,515	2	5	23	178,696	175,818		2	2	235,253	235,253	1	1	20	1,315,139
North Dakota	174,102	172,895	4			604,585	591,648	6			223,120	223,120	2			636,650
Ohio	1,544,019	1,528,804	8	1	6	2,767,792	2,444,070	10		2	405,140	201,010	1		7	1,697,332
Oklahoma	181,964	178,170	1		30	946,514	943,104	7		6	266,890	228,853	2		3	1,558,730
Oregon	419,536	355,255	4		3	203,552	187,715	2			2,733	2,733			2	442,391
Pennsylvania	1,675,339	1,675,011	13	1		3,232,872	3,187,962	15			359,074	359,074	2			2,542,633
Rhode Island	205,241	205,241		1		3,655	3,655									273,921
South Carolina	347,737	334,405	6		21	245,569	224,728	2	3	2	238,868	123,735	1	2	8	977,579
South Dakota	507,443	507,443	13		9	521,843	505,893	9			41,200	41,200	2			827,558
Tennessee	301,580	289,686	3	3		1,325,647	1,325,647	7	1		102,106	102,106		1	2	1,005,387
Texas	1,225,529	1,204,217	15			1,467,237	1,455,643	14		4	4,350	4,350			1	2,372,045
Utah	58,794	58,052	2		19	69,369	69,369	1		9	60,140	60,140			23	359,438
Vermont	18,683	18,671			4	322,869	293,090	2								103,978
Virginia	438,014	438,014	2	2	2	442,273	422,313	4	3	1	33,021	33,021	1		2	959,061
Washington	175,020	171,817	1	2	2	1,257,474	560,800	7		1	5,192	5,192			2	439,527
West Virginia	253,143	247,512	3		6	654,510	654,510	6	1	2	3,330	3,330			1	776,753
Wisconsin	480,777	451,749	2		39	568,623	567,635	3	2	6	15,484	15,484			3	1,673,330
Wyoming	483,177	466,619	5		1	1,974	1,974				8,416	8,416			6	429,924
District of Columbia	2,193	2,193				299,675	275,206		1							103,351
Hawaii	137,618	187,618	2			214,170	213,655	2								282,756
Puerto Rico	103,629	102,980	1			780,619	772,706	11								363,914
TOTALS	21,868,772	21,226,775	158	52	403	35,948,190	33,568,399	253	49	126	8,235,172	7,521,614	37	17	294	48,323,014

PUBLIC ROADS

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FEDERAL WORKS AGENCY
PUBLIC ROADS ADMINISTRATION

VOL. 23, NO. 2 APRIL 1942

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PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 23, No. 2 April 1942

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Commissioner of Public Roads, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

SOME CHARACTERISTICS OF MOTOR-VEHICLE TRAVEL

BY THE DIVISION OF CONTROL, PUBLIC ROADS ADMINISTRATION

Reported by ROBERT H. PADDOCK, Highway Engineer-Economist

There is much current interest in the purpose of travel on the Nation's highway and street systems. The accompanying article is timely both because of existing war-impelled restrictions on the production and sale of motor vehicles and tires for private use and because of the recurring threat of motor fuel rationing.

Road-use studies of the highway planning surveys in 35 States indicate that 58.6 percent of passenger-car travel in those States was for business purposes and that nearly all truck travel was for business or commercial purposes. Thus, during the period here reported, 65 percent of the annual travel of all motor vehicles was for business purposes.

The article also presents an estimate of total vehicle-miles traveled during 1940 and, on the assumption that no appreciable change occurred between the study years (usually 1936–37) and 1940 in the ratio of business to social-recreational travel, the vehicle-mile total for 1940 by type of vehicle is given in terms of the estimated extent of driving in connection with the two major travel purposes.

The age of a passenger car is shown to exert an influence upon the character of its use to such an extent that, although the number of round trips made annually averages 619 for vehicles 1 year old and 417 for vehicles more than 8 years old, the percentage of total travel for business purposes was 55 percent for the former and 61 percent for the latter. Since a relationship has previously been shown to exist between situs of vehicle ownership and age of vehicle, the greater percentage use of the rural-owned vehicle for business purposes is a corollary.

APPROXIMATELY 65 percent of all motor-vehicle travel in the United States is for "business" purposes, or in connection with such essential activities as earning a living or maintaining the home. Data on this subject were obtained in the road-use studies conducted as a part of the State-wide highway planning surveys that were initiated in 1935.

The road-use studies are integral parts of the highway planning surveys that have been undertaken in all States and the District of Columbia. Information regarding road use was obtained by means of a large number of personal interviews with motor-vehicle owners and drivers. These interviews were selected and carefully analyzed to provide a proper representation of each geographical division of a State, of each group of governmental jurisdictions within similar population ranges, of various occupations, and of vehicles in operation according to types and ages.

Some of the preliminary results of the studies, relating particularly to use of the various highway systems and length of trips, have already been published.[1] The application of the road-use survey methods to other problems has also been studied [2] and considerable statistical review and analysis of the road-use methods and data have been made.[3]

Two general methods of obtaining the information were used. In most States a group of full-time, trained personnel interviewed motor-vehicle owners to obtain the desired information. In other States the data were obtained by high school students from their parents and friends who owned motor vehicles. The students received instruction regarding the data to be obtained from a small group of full-time instructors who visited all the high schools in the State. Figure 1 shows the methods by which road-use information has been obtained in the various States. Tabulation and analysis of the data have not yet been completed for Connecticut, Delaware, Georgia, Maine, Massachusetts, Mississippi, New Jersey, Rhode Island, and Tennessee. The Pennsylvania survey did not provide for a classification of business and pleasure travel.

The primary purpose of the road-use surveys was to obtain information that would make it possible to estimate:

1. The total amount of travel on the various highway systems in a given area or in the State; and
2. The amount of travel performed on the various highway systems in the State by vehicle owners residing in the several governmental jurisdictions.

On the basis of these estimates it would be possible to compare the travel benefits received by the various population groups of the State with their respective highway-user tax contributions.

TRAVEL DATA DIVIDED INTO BUSINESS AND PLEASURE CLASSIFICATIONS

However, in addition to such basic data it was possible to obtain considerable supplemental information from the road-use interviews. The classification of travel into business and pleasure purposes was one type of supplemental information. Figure 2 is a reproduction of the front of the Connecticut interview form and shows how the data were originally entered on each interview so that the separation into business and pleasure travel could be made in the office analyses. In the initial surveys only two classifications were used—business and pleasure. Subsequently, it was believed that the pleasure travel might be more properly classified as recreational and social travel and this separation is made in the Connecticut and other recent surveys.

Generally the classification of travel into business and pleasure purposes was applied only to the analysis of passenger-car travel. For comparative purposes in this study the travel of all trucks has been assumed to be entirely for business purposes. It is known that a small percentage of truck travel is for purposes other than business, but this is usually limited to farmers' trucks that are often used as general utility vehicles. In a few States where truck travel was also classified

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[1] A description of the road-use survey methods is given in Preliminary Results of Road-Use Studies, by Robert H. Paddock and Roe P. Rodgers, PUBLIC ROADS, May 1939.

[2] The Application of Road-Use Survey Methods in Traffic Origin and Destination Analysis, by T. M. C. Martin and Homer L. Baker, PUBLIC ROADS, May 1941.

[3] Road-use studies have been conducted in all but four States and the District of Columbia, and are now under way in Delaware. Most of the surveys were made in 1936 and 1937 although several have been made more recently and the New York study was conducted in 1934–35. More than 800,000 usable interviews have been obtained in the several States. Table 1 shows the number of interviews obtained in each of the States in which the surveys have been conducted.

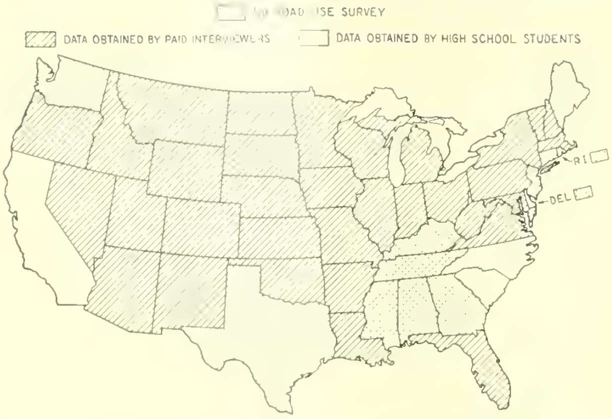

NO ROAD USE SURVEY
DATA OBTAINED BY PAID INTERVIEWERS
DATA OBTAINED BY HIGH SCHOOL STUDENTS
R.I.
DEL

FIGURE 1.—METHODS BY WHICH ROAD-USE INFORMATION HAS BEEN OBTAINED IN THE VARIOUS STATES.

TABLE 1.—*States in which road-use interviews were obtained*

State	Interviews obtained for—		
	Passenger cars	Trucks	All vehicles
Alabama	14,600	2,989	17,589
Arizona	3,440	2,277	5,717
Arkansas	4,176	1,992	6,168
Colorado	1,892	1,262	3,154
Connecticut	54,194	5,021	59,215
Florida	6,972	3,203	10,175
Georgia	25,912	5,667	31,579
Idaho	2,000	1,000	3,000
Illinois	29,236	10,533	39,769
Indiana	13,567	6,264	19,831
Iowa	8,066	3,314	11,380
Kansas	9,244	3,588	12,832
Kentucky	15,290	2,360	17,650
Louisiana	4,614	2,624	7,238
Maine	14,000	2,800	16,800
Maryland	8,616	2,000	10,616
Massachusetts	20,379	9,830	30,209
Michigan	25,300	6,313	31,613
Minnesota	13,657	7,736	21,393
Mississippi	7,017	1,525	8,542
Missouri	12,445	5,770	18,215
Montana	2,598	1,887	4,485
Nebraska	8,213	5,077	13,290
Nevada	2,244	990	3,234
New Hampshire	2,007	1,039	3,046
New Jersey	63,000	9,000	72,000
New Mexico	2,761	2,463	5,224
New York	20,000	5,000	25,000
North Dakota	2,550	1,004	3,554
Ohio	33,761	9,419	43,180
Oklahoma	10,996	3,743	14,739
Oregon	5,020	2,277	7,297
Pennsylvania	23,600	10,640	34,240
Rhode Island	9,673	2,416	12,089
South Dakota	3,607	1,750	5,357
Tennessee	21,600	2,400	24,000
Texas	71,500	20,500	92,000
Utah	2,186	1,135	3,321
Vermont	1,494	873	2,367
Virginia	6,495	3,241	9,736
Washington	18,828	3,118	21,946
West Virginia	4,265	1,867	6,132
Wisconsin	15,041	7,059	22,100
Wyoming	1,258	803	2,061
Total	627,314	185,769	813,083

as to purpose it was found that the use of trucks for other than business purposes was about 2 to 3 percent of the total truck travel.

Analysis of truck travel according to business or pleasure use was made in 15 States. A summary of the results is given in table 2. For the 15 States, 2.5 percent of truck travel was for other than business purposes. It will be noted that these 15 States include no highly industrialized or urbanized States such as New York, Pennsylvania, New Jersey, Ohio, Michigan, or Illinois. One of the fifteen States is a Southern State and eight are Western States. In the represented Middle States, where a moderate industrialization and urbanization is combined with agricultural development, the percentage of use for other than business purposes is only 0.7 percent for Indiana and 0.8 percent for Minnesota and Wisconsin.

It is believed that the percentages shown in table 2 are not entirely representative of the country as a whole and the data for the 15 States are not sufficient to provide a basis for estimating percentages for the other States. For the purposes of this study it has been considered practicable to classify all truck travel as having been made for business purposes. It will be noted in table 2 that only in the unincorporated areas for the 15 States reported was there much use of trucks for other purposes. The total in this group for the 15 States was 4.4 percent for pleasure purposes. In the next group, incorporated places having a population of 1,000 or less, the percentage was only 2.1 and in each of the other population groups the percentage was less than 2.0.

In the road-use surveys initiated during 1935 and 1936 travel of passenger cars was classified as follows:

Business use.—Trips to regular business or work, other business trips, hauling milk, farm produce, etc., trips to market, to the railroad or bus station, shopping trips, and taking children to and from school.

Pleasure use.—All other travel such as going to games, week-end vacation trips, hunting, fishing, and holiday trips, going to the theater, dancing or visiting, and Sunday and evening drives.

For surveys initiated in 1937 and in subsequent years the following classification of trips was used:

Business
To business or work.
Business trips.
Hauling milk, etc.
To market.
To railroad or bus station.
Shopping.
Deliveries.

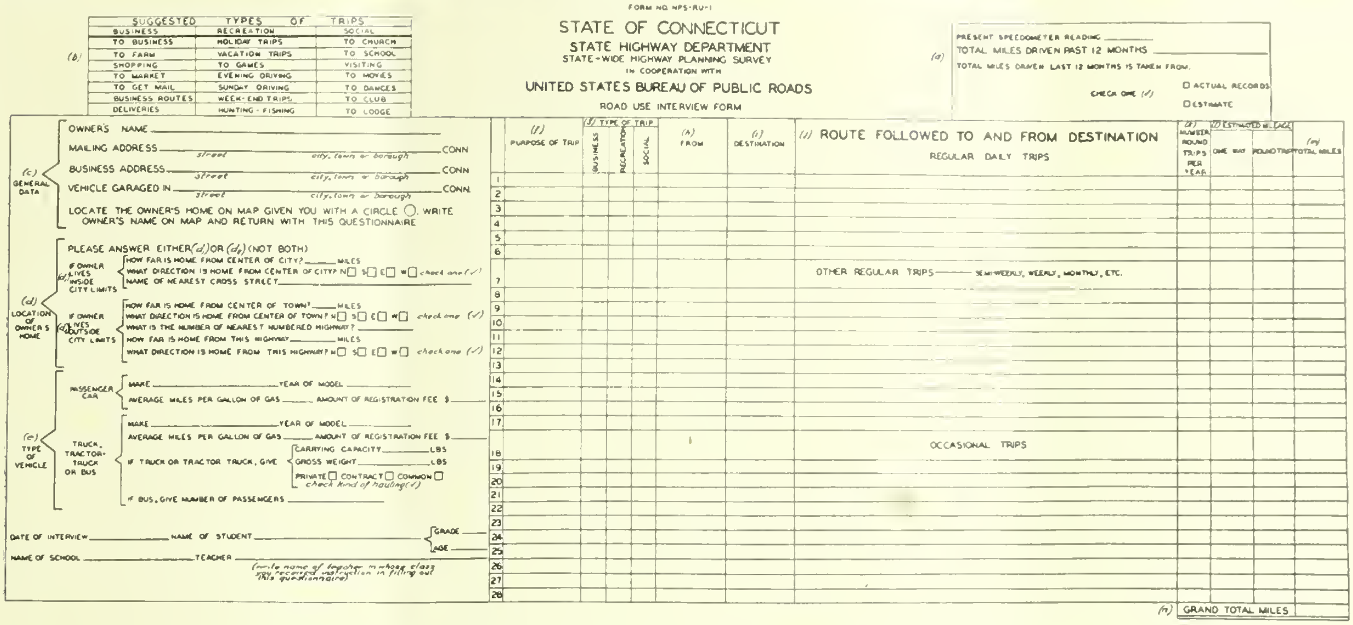
Social
Children to and from school.
To church.
To theater.
To dances.
Visiting.

Recreational
To games.
Sunday drives.
Evening drives.
Week end.
Holiday.
Hunting.
Fishing.

It will be noted that there was no change in the business classification except that "children to and from school" was placed under the "social" classification. Subsequently special studies of this particular item have been made and indicate that less than 2 percent of the total travel is for that purpose and that this change in classification does not affect the general relations existing between business and pleasure travel.

FIFTY-SEVEN PERCENT OF PASSENGER-CAR TRAVEL FOR BUSINESS PURPOSES

Tables 3 and 4 present the general data available from the road-use surveys for the percentage classification of passenger-car travel into business and pleasure purposes by States and by population groups. The classification of travel by population groups refers to the travel performed by motor-vehicle owners resident in the several population groupings and does not refer to the travel which occurred on the streets of those places. Table 3 shows the percentages of travel by residents of 35 States in the respective States of residence, while table 4 shows the percentages of travel in all States by residents of



FORM NO. NPS-RU-1

STATE OF CONNECTICUT
STATE HIGHWAY DEPARTMENT
STATE-WIDE HIGHWAY PLANNING SURVEY
IN COOPERATION WITH
UNITED STATES BUREAU OF PUBLIC ROADS
ROAD USE INTERVIEW FORM

(a)
PRESENT SPEEDOMETER READING ________
TOTAL MILES DRIVEN PAST 12 MONTHS ________
TOTAL MILES DRIVEN LAST 12 MONTHS IS TAKEN FROM.
CHECK ONE (✓) ☐ ACTUAL RECORDS ☐ ESTIMATE

(b)

SUGGESTED TYPES OF TRIPS		
BUSINESS	RECREATION	SOCIAL
TO BUSINESS	HOLIDAY TRIPS	TO CHURCH
TO FARM	VACATION TRIPS	TO SCHOOL
SHOPPING	TO GAMES	VISITING
TO MARKET	EVENING DRIVING	TO MOVIES
TO GET MAIL	SUNDAY DRIVING	TO DANCES
BUSINESS ROUTES	WEEK-END TRIPS	TO CLUB
DELIVERIES	HUNTING - FISHING	TO LODGE

(c) GENERAL DATA
OWNER'S NAME ________
MAILING ADDRESS ________ street ________ city, town or borough CONN
BUSINESS ADDRESS ________ street ________ city, town or borough CONN
VEHICLE GARAGED IN ________ street ________ city, town or borough CONN
LOCATE THE OWNER'S HOME ON MAP GIVEN YOU WITH A CIRCLE ○. WRITE OWNER'S NAME ON MAP AND RETURN WITH THIS QUESTIONNAIRE

(d) LOCATION OF OWNER'S HOME
PLEASE ANSWER EITHER (d₁) OR (d₂) (NOT BOTH)
(d₁) IF OWNER LIVES INSIDE CITY LIMITS
HOW FAR IS HOME FROM CENTER OF CITY? ____ MILES
WHAT DIRECTION IS HOME FROM CENTER OF CITY? N☐ S☐ E☐ W☐ check one (✓)
NAME OF NEAREST CROSS STREET ________
(d₂) IF OWNER LIVES OUTSIDE CITY LIMITS
HOW FAR IS HOME FROM CENTER OF TOWN? ____ MILES
WHAT DIRECTION IS HOME FROM CENTER OF TOWN? N☐ S☐ E☐ W☐ check one (✓)
WHAT IS THE NUMBER OF NEAREST NUMBERED HIGHWAY? ________
HOW FAR IS HOME FROM THIS HIGHWAY ________ MILES
WHAT DIRECTION IS HOME FROM THIS HIGHWAY? N☐ S☐ E☐ W☐ check one (✓)

(e) TYPE OF VEHICLE
PASSENGER CAR
MAKE ________ YEAR OF MODEL ________
AVERAGE MILES PER GALLON OF GAS ____ AMOUNT OF REGISTRATION FEE $ ____
TRUCK, TRACTOR-TRUCK OR BUS
MAKE ________ YEAR OF MODEL ________
AVERAGE MILES PER GALLON OF GAS ____ AMOUNT OF REGISTRATION FEE $ ____
IF TRUCK OR TRACTOR TRUCK, GIVE
CARRYING CAPACITY ________ LBS
GROSS WEIGHT ________ LBS
PRIVATE ☐ CONTRACT ☐ COMMON ☐ check kind of hauling (✓)
IF BUS, GIVE NUMBER OF PASSENGERS ________

DATE OF INTERVIEW ________ NAME OF STUDENT ________ GRADE ____ AGE ____
NAME OF SCHOOL ________ TEACHER ________ (write name of teacher in whose class you received instruction in filling out this questionnaire)

	(f) PURPOSE OF TRIP	(g) TYPE OF TRIP: BUSINESS	RECREATION	SOCIAL	(h) FROM	(i) DESTINATION	(j) ROUTE FOLLOWED TO AND FROM DESTINATION REGULAR DAILY TRIPS	(k) NUMBER ROUND TRIPS PER YEAR	(l) ESTIMATED MILEAGE: ONE WAY	ROUND TRIP	(m) TOTAL MILES
1											
2											
3											
4											
5											
6											
7							OTHER REGULAR TRIPS——— SEMI-WEEKLY, WEEKLY, MONTHLY, ETC.				
8											
9											
10											
11											
12											
13											
14											
15											
16											
17											
18							OCCASIONAL TRIPS				
19											
20											
21											
22											
23											
24											
25											
26											
27											
28											
								(n) GRAND TOTAL MILES			

FIGURE 2.—INTERVIEW FORM USED IN CONNECTICUT ROAD-USE STUDY.

TABLE 2.—*Classification of the total travel of trucks of 15 States for business and pleasure purposes*

State	Distribution of travel by residents of—																	
	Unincorporated areas		Incorporated places having a population of—														All places	
			1,000 or less		1,001 to 2,500		2,501 to 5,000		5,001 to 10,000		10,001 to 25,000		25,001 to 100,000		100,001 or more			
	Travel for—		Travel for—		Travel for—		Travel for—		Travel for—		Travel for—		Travel for—		Travel for—		Travel for—	
	Business	Pleasure	Business	Pleasure	Business	Pleasure	Business	Pleasure	Business	Pleasure	Business	Pleasure	Business	Pleasure	Business	Pleasure	Business	Pleasure
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Alabama	90.1	9.9	96.5	3.5	96.9	3.1	97.5	2.5	95.4	4.6	97.8	2.2	99.3	0.7	97.9	2.1	94.2	5.8
Arizona	96.0	4.0	96.0	4.0	97.6	2.4	97.7	2.3	99.0	1.0	-------	-------	99.6	.4	-------	-------	98.0	2.0
Indiana	98.4	1.6	98.8	1.2	99.4	.6	99.1	.9	99.3	.7	99.7	.3	99.9	.1	99.8	.2	99.3	.7
Kentucky	94.2	5.8	95.6	4.4	96.8	3.2	96.7	3.3	97.2	2.8	91.4	8.6	96.4	3.6	98.1	1.9	95.4	4.6
Minnesota	98.3	1.7	98.9	1.1	99.7	.3	100.0	(1)	98.6	1.4	99.8	.2	-------	-------	99.8	.2	99.2	.8
Missouri	97.0	3.0	97.6	2.4	97.6	2.4	99.2	.8	99.2	.8	99.3	.7	98.4	1.6	99.2	.8	98.4	1.6
Nebraska	98.7	1.3	99.7	.3	99.7	.3	99.5	.5	99.9	.1	99.9	.1	99.9	.1	99.6	.4	99.5	.5
Nevada	94.6	5.4	96.4	3.6	99.3	.7	99.4	.6	99.5	.5	98.9	1.1	-------	-------	-------	-------	97.4	2.6
New Mexico	95.7	4.3	98.6	1.4	98.9	1.1	99.2	.8	99.9	.1	99.6	.4	100.0	(1)	-------	-------	97.9	2.1
North Dakota	97.0	3.0	96.7	3.3	98.7	1.3	100.0	-------	98.6	1.4	99.6	.4	100.0	-------	-------	-------	97.8	2.2
Oklahoma	94.7	5.3	94.7	5.3	94.1	5.9	96.4	3.6	94.5	5.5	92.9	7.1	96.4	3.6	95.3	4.7	94.7	5.3
Oregon	86.5	13.5	86.5	13.5	94.2	5.8	93.6	6.4	97.1	2.9	93.4	6.6	94.0	6.0	96.1	3.9	91.5	8.5
Vermont	96.8	3.2	98.1	1.9	98.5	1.5	98.2	1.8	98.2	1.8	99.3	.7	-------	-------	-------	-------	97.6	2.4
Washington	91.9	8.1	95.6	4.4	98.0	2.0	96.5	3.5	96.5	3.5	97.1	2.9	93.2	6.8	97.8	2.2	95.2	4.8
Wisconsin	98.5	1.5	99.6	.4	99.6	.4	99.6	.4	99.6	.4	99.7	.3	99.5	.5	99.7	.3	99.2	.8
Total	95.6	4.4	97.9	2.1	98.2	1.8	98.5	1.5	98.4	1.6	98.1	1.9	99.0	1.0	98.8	1.2	97.5	2.5

[1] Less than 0.1 percent.

31 States. Because of the methods of tabulation adopted by the States, data for Arkansas, Colorado, Michigan, and New York are available only for the travel within the State of residence as shown in table 3.

Comparison of the data in tables 3 and 4 indicates that in all but one State the percentage of total travel for business purposes is smaller than the percentage of the travel within the State of residence for business purposes. This indicates that generally passenger-car travel outside the State of residence is predominantly for pleasure purposes, probably for vacation or holiday trips.

The one State where this condition is not true is Utah. The factor that controls this State percentage is the travel of Ogden and Salt Lake City motor-vehicle owners. The out-of-State travel of the motor-vehicle owners living in unincorporated areas was also largely for business purposes and may be accounted for by the heavy concentration of residents in the northern sections of the State whose business centers are towns in Idaho and Wyoming. In 4 other States individual population groups show this difference, but in all cases the differences in percentages are small and in no case are large enough to affect the State total. In Montana, for incorporated places having a population of less than 1,000 there is a difference of 0.2 percent. In Louisiana, for incorporated places having a population of 1,001 to 2,500 there is a difference of 0.1 percent. In New Mexico, for incorporated places having a population of 2,501 to 5,000 there is a difference of 3.2 percent. This may be explained by the fact that all of the incorporated places in this group except Las Vegas are near

TABLE 3.—*Classification of the travel in the State of residence of passenger cars of 35 States*

State	Distribution of travel by residents of—																	
	Unincorporated areas		Incorporated places having a population of—														All places	
			1000 or less		1,001 to 2,500		2,501 to 5,000		5,001 to 10,000		10,001 to 25,000		25,001 to 100,000		100,001 or more			
	Travel for—		Travel for—		Travel for—		Travel for—		Travel for—		Travel for—		Travel for—		Travel for—		Travel for—	
	Business	Pleasure	Business	Pleasure	Business	Pleasure	Business	Pleasure	Business	Pleasure	Business	Pleasure	Business	Pleasure	Business	Pleasure	Business	Pleasure
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Alabama	73.6	26.4	73.6	26.4	72.9	27.1	66.0	34.0	66.1	33.9	66.7	33.3	68.4	31.6	65.0	35.0	70.3	29.7
Arizona	68.3	31.7	66.5	33.5	62.4	37.6	62.3	37.7	58.1	41.9			60.8	39.2			64.1	35.9
Arkansas	79.6	20.4	78.9	21.1	72.3	27.7	69.1	30.9	61.8	38.2	66.6	33.4	66.0	34.0			73.0	27.0
Colorado	73.7	26.3	63.7	36.3	62.5	37.5	55.1	44.9	63.5	36.5	62.3	37.7	51.1	48.9	58.4	41.6	62.6	37.4
Florida	69.6	30.4	71.9	28.1	64.7	35.3	59.1	40.9	61.2	38.8	50.5	49.5	50.8	49.2	54.0	46.0	59.5	40.5
Idaho	75.3	24.7	70.2	29.8	59.3	40.7	63.4	36.6	64.5	35.5	59.6	40.4					69.1	30.9
Illinois	61.9	38.1	53.7	46.3	49.3	50.7	48.5	51.5	48.5	51.5	43.8	56.2	47.3	52.7	54.8	45.2	52.8	47.2
Indiana	66.9	33.1	67.2	32.8	56.1	43.9	53.9	46.1	47.0	53.0	55.3	44.7	50.1	49.9	49.9	50.1	57.3	42.7
Iowa	67.7	32.3	63.9	36.1	54.9	45.1	51.7	48.3	31.6	68.4	45.1	54.9	62.6	37.4	51.7	48.3	59.4	40.6
Kansas	69.6	30.4	58.1	41.9	49.8	50.2	53.9	46.1	53.9	46.1	55.9	44.1	53.8	46.2	54.2	45.8	59.2	40.8
Kentucky	67.7	32.3	69.2	30.8	68.2	31.8	65.3	34.7	66.5	33.5	69.8	30.2	62.6	37.4	64.3	35.7	66.7	33.3
Louisiana	81.6	18.4	72.7	27.3	76.1	23.9	65.1	34.9	65.4	34.6	70.9	29.1	71.4	28.6	58.7	41.3	71.2	28.8
Maryland	74.7	25.3	65.9	34.1	69.3	30.7	64.0	36.0	64.0	36.0	68.4	31.6	57.9	42.1	79.7	20.3	74.7	25.3
Michigan	61.9	38.1	52.8	47.2	49.9	50.1	54.9	45.1	49.9	50.1	53.7	46.3	50.2	49.8	52.2	47.8	53.4	46.6
Minnesota	59.4	40.6	47.8	52.2	41.8	58.2	43.5	56.5	40.4	59.6	45.0	55.0			39.8	60.2	46.7	53.3
Missouri	75.1	24.9	69.4	30.6	69.4	30.6	63.9	36.1	63.9	36.1	55.5	44.5	67.7	32.3	60.1	39.9	65.3	34.7
Montana	73.0	27.0	51.2	48.8	55.7	44.3	63.3	36.7	53.7	46.3	56.2	43.8	51.4	48.6			63.5	36.5
Nebraska	71.2	28.8	65.8	34.2	59.7	40.3	59.8	40.2	60.8	39.2	61.1	38.9	57.2	42.8	63.0	37.0	65.1	34.9
Nevada	76.8	23.2	55.5	44.5	73.0	27.0	64.9	35.1	71.0	29.0	59.4	40.6					70.0	30.0
New Hampshire			72.4	27.6	70.2	29.8	63.7	36.3	63.7	36.3	53.5	46.5	52.5	47.5			62.8	37.2
New Mexico	57.7	42.3	68.0	32.0	66.7	33.3	38.4	61.6	60.6	39.4	56.4	43.6	59.0	41.0			58.0	42.0
New York	57.1	42.9	54.1	45.9	54.1	45.9	49.2	50.8	49.2	50.8	50.1	49.9	49.9	50.1	57.5	42.5	55.8	44.2
North Dakota	70.9	29.1	62.5	37.5	67.2	32.8	59.5	40.5	66.3	33.7	57.4	42.6	66.3	33.7			67.4	32.6
Ohio	59.4	40.6	56.7	43.3	52.7	47.3	44.2	55.8	44.2	55.8	44.5	55.5	48.4	51.6	49.3	50.7	51.3	48.7
Oklahoma	79.2	20.8	79.2	20.8	73.5	26.5	67.9	32.1	78.9	21.1	70.1	29.9	65.8	34.2	62.8	37.2	72.6	27.4
Oregon	67.1	32.9	58.4	41.6	48.7	51.3	56.2	43.8	48.1	51.9	53.0	47.0	38.1	61.9	54.6	45.4	57.9	42.1
South Dakota	66.8	33.2	57.6	42.4	52.7	47.3	57.0	43.0	54.1	45.9	55.8	44.2	52.5	47.5			60.0	40.0
Texas	63.1	36.9	66.0	34.0	61.0	39.0	61.2	38.8	57.4	42.6	55.5	44.5	56.5	43.5	57.3	42.7	59.9	40.1
Utah	51.5	48.5	51.1	48.9	55.9	44.1	53.4	46.6	49.4	50.6	32.2	67.8	47.4	52.6	61.2	38.8	54.4	45.6
Vermont	70.6	29.4	71.2	28.8	66.7	33.3	58.0	42.0	58.0	42.0	58.4	41.6					65.6	34.4
Virginia	70.2	29.8	58.4	41.6	74.5	25.5	56.9	43.1	61.0	39.0	65.2	34.8	57.3	42.7	51.8	48.2	65.7	34.3
Washington	69.7	30.3	65.2	34.8	64.6	35.4	61.6	38.4	61.6	38.4	59.8	40.2	58.3	41.7	61.9	38.1	64.3	35.7
West Virginia	64.1	35.9	62.3	37.7	54.9	45.1	61.2	38.8	52.9	47.1	47.1	52.9	53.4	46.6			58.5	41.5
Wisconsin	55.2	44.8	53.6	46.4	48.6	51.4	49.9	50.1	53.6	46.4	48.7	51.3	52.9	47.1	43.8	56.2	50.8	49.2
Wyoming	70.1	29.9	59.1	40.9	50.9	49.1	49.4	50.6	45.0	55.0	58.2	41.8					59.7	40.3
Total	67.0	33.0	61.0	39.0	57.7	42.3	56.0	44.0	54.6	45.4	53.9	46.1	53.7	46.3	54.5	45.5	58.6	41.4

the boundaries of the State. For several of them El Paso, Tex., is a natural shopping and marketing center and undoubtedly is the cause of the increased percentage of out-of-State travel for business purposes.

In Ohio, for incorporated places having a population of 5,001 to 10,000, there is a difference of 2.3 percent. There is no apparent geographic reason for this condition in Ohio since all the places in this group are spread more or less uniformly throughout the State.

Classification of the data of table 4 by geographic groupings of the States (table 5) indicates that in the Middle States the percentage of travel for business purposes is definitely lower than in other sections of the country. For purposes of comparison the groupings of States is the one that has previously been used in studying motor-vehicle trends in the United States.[4]

The groupings used are:

Region	States
Northeast	Maine, New Hampshire, Vermont, Massachusetts, Rhode Island, Connecticut, New York, Pennsylvania, New Jersey, Delaware and Maryland.
Southeast	Virginia, West Virginia, Kentucky, North Carolina, Tennessee, South Carolina, Georgia, Florida, Alabama, Mississippi, Louisiana, and Arkansas.
Middle States	Ohio, Michigan, Indiana, Illinois, Wisconsin, Missouri, Iowa, and Minnesota.
Northwest	North Dakota, South Dakota, Nebraska, Kansas, Montana, Wyoming, Colorado, Idaho, and Utah.
Southwest	Oklahoma, Texas, New Mexico, and Arizona.
Far West	Washington, Oregon, California, and Nevada.

Data are available for a representative number of States in all except the Northeast region, where data for only Vermont, New Hampshire, and Maryland, are available. In a regrouping of this region with the Middle States with which it is somewhat similar it is found that the percentage of travel for business purposes (52.5 percent) is lower than in any other region or group of regions. Even if New York data, which are only for travel within the State of residence, were included the percentage of travel for business purposes in this region would still be lower than in the other areas since New York travel for business purposes was 55.4 percent of the total travel (table 3).

TWO-THIRDS OF ALL MOTOR-VEHICLE TRAVEL FOR BUSINESS PURPOSES IN 1940

Table 5 also indicates that generally as the size of the incorporated place increases the percentage of the motor-vehicle travel of the residents of those places for business purposes decreases. This condition appears to be true for all places except the cities having more than 100,000 population. In the Northeast region, Balti-

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[4] Significant Trends in Motor-Vehicle Registrations and Receipts, by Robert H. Paddock, PUBLIC ROADS, October 1939.

TABLE 4.—*Classification of the total travel of passenger cars of 31 States*

State	Distribution of travel by residents of—															
	Unincorporated areas		Incorporated places having a population of—												All places	
			1,000 or less		1,001 to 2,500		2,501 to 5,000		5,001 to 10,000		10,001 to 25,000		25,001 to 100,000		100,001 or more		
	Travel for—		Travel for—		Travel for—		Travel for—		Travel for—		Travel for—		Travel for—		Travel for—		Travel for—	
	Business	Pleasure	Business	Pleasure	Business	Pleasure	Business	Pleasure	Business	Pleasure	Business	Pleasure	Business	Pleasure	Business	Pleasure	Business	Pleasure
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Alabama	72.6	27.4	72.6	27.4	70.5	29.5	64.2	35.8	63.9	36.1	64.3	35.7	67.6	32.4	63.1	36.9	68.9	31.1
Arizona	62.6	37.4	59.1	40.9	56.6	43.4	58.6	41.4	56.5	43.5	-----	-----	55.6	44.4	-----	-----	59.1	40.9
Florida	68.2	31.8	70.4	29.6	62.3	37.7	57.2	42.8	58.0	42.0	48.0	52.0	48.6	51.4	52.0	48.0	57.4	42.6
Idaho	73.2	26.8	69.1	30.9	55.5	44.5	61.7	38.3	63.2	36.8	56.9	43.1	-----	-----	-----	-----	66.8	33.2
Illinois	60.3	39.7	51.6	48.4	47.4	52.6	45.4	54.6	45.4	54.6	40.4	59.6	43.4	56.6	47.9	52.1	48.1	51.9
Indiana	65.3	34.7	65.1	34.9	54.6	45.4	51.9	48.1	46.3	53.7	52.8	47.2	46.3	53.7	46.8	53.2	54.8	45.2
Iowa	67.4	32.6	61.8	38.2	53.2	46.8	49.8	50.2	29.3	70.7	43.3	56.7	61.6	38.4	48.7	51.3	57.8	42.2
Kansas	67.1	32.9	55.6	44.4	46.4	53.6	50.8	49.2	50.8	49.2	54.1	45.9	51.3	48.7	50.3	49.7	56.2	43.8
Kentucky	66.2	33.8	68.1	31.9	65.8	34.2	62.2	37.8	64.1	35.9	67.9	32.1	59.8	40.2	61.3	38.7	64.6	35.4
Louisiana	80.8	19.2	71.2	28.8	76.2	23.8	64.2	35.8	64.8	35.2	70.6	29.4	70.1	29.9	57.9	42.1	70.1	29.9
Maryland	72.5	27.5	62.2	37.8	66.0	34.0	60.0	40.0	60.0	40.0	67.1	32.9	54.1	45.9	75.6	24.4	71.4	28.6
Minnesota	58.5	41.5	46.7	53.3	41.2	58.8	42.2	57.8	39.7	60.3	43.8	56.2	-----	-----	39.3	60.7	45.8	54.2
Missouri	73.6	26.4	67.9	32.1	67.9	32.1	62.3	37.7	62.3	37.7	53.4	46.6	65.1	34.9	57.2	42.8	63.1	36.9
Montana	72.7	27.3	51.4	48.6	54.1	45.9	61.2	38.8	53.5	46.5	55.7	44.3	51.3	48.7	-----	-----	63.0	37.0
Nebraska	69.4	30.6	62.2	37.8	55.9	44.1	55.1	44.9	57.5	42.5	58.0	42.0	54.7	45.3	60.6	39.4	62.3	37.7
Nevada	72.6	27.4	51.8	48.2	70.5	29.5	61.2	38.8	67.9	32.1	55.1	44.9	-----	-----	-----	-----	65.9	34.1
New Hampshire	-----	-----	70.6	29.4	67.0	33.0	61.8	38.2	61.8	38.2	49.6	50.4	48.3	51.7	-----	-----	59.5	40.5
New Mexico	55.6	44.4	62.0	38.0	63.9	36.1	41.6	58.4	58.6	41.4	52.2	47.8	55.6	44.4	-----	-----	55.6	44.4
North Dakota	67.8	32.2	57.4	42.6	60.0	40.0	54.2	45.8	60.5	39.5	52.3	47.7	51.5	48.5	-----	-----	62.4	37.6
Ohio	58.4	41.6	55.3	44.7	50.5	49.5	46.5	53.5	46.5	53.5	42.6	57.4	46.2	53.8	46.1	53.9	49.4	50.6
Oklahoma	77.4	22.6	77.4	22.6	72.5	27.5	65.7	34.3	77.2	22.8	68.6	31.4	63.7	36.3	61.0	39.0	70.8	29.2
Oregon	64.3	35.7	55.8	44.2	46.8	53.2	53.5	46.5	45.7	54.3	51.4	48.6	37.3	62.7	52.4	47.6	55.4	44.6
South Dakota	64.7	35.3	55.1	44.9	49.1	50.9	54.2	45.8	49.3	50.7	52.6	47.4	48.0	52.0	-----	-----	56.9	43.1
Texas	62.4	37.6	65.1	34.9	59.8	40.2	59.7	40.3	56.6	43.4	54.3	45.7	54.9	45.1	56.0	44.0	58.7	41.3
Utah	53.1	46.9	50.3	49.7	55.3	44.7	50.6	49.4	43.8	56.2	32.0	68.0	49.7	50.3	62.9	37.1	55.1	44.9
Vermont	67.5	32.5	67.5	32.5	64.8	35.2	53.6	46.4	53.6	46.4	55.1	44.9	-----	-----	-----	-----	62.3	37.7
Virginia	68.5	31.5	57.4	42.6	72.8	27.2	55.5	44.5	59.5	40.5	64.2	35.8	56.1	43.9	49.4	50.6	64.0	36.0
Washington	67.8	32.2	63.2	36.8	61.5	38.5	58.4	41.6	58.4	41.6	57.3	42.7	56.3	43.7	59.9	40.1	62.1	37.9
West Virginia	62.3	37.7	59.5	40.5	52.9	47.1	58.2	41.8	49.9	50.1	44.6	55.4	50.2	49.8	-----	-----	55.8	44.2
Wisconsin	54.4	45.6	52.7	47.3	47.4	52.6	48.7	51.3	52.8	47.2	47.8	52.2	51.0	49.0	42.4	57.6	49.6	50.4
Wyoming	67.0	33.0	58.2	41.8	47.8	52.2	48.4	51.6	43.4	56.6	52.9	47.1	-----	-----	-----	-----	56.4	43.6
Total	65.6	34.4	59.5	40.5	56.4	43.6	53.9	46.1	53.1	46.9	51.4	48.6	51.5	48.5	51.5	48.5	56.7	43.3

TABLE 5.—*Percentage of total passenger-car travel for business purposes, by regions*

Region [1]	Percentage of total travel for business by residents of—								
	Unincorporated areas	Incorporated places having a population of—							All places
		1,000 or less	1,001 to 2,500	2,501 to 5,000	5,001 to 10,000	10,001 to 25,000	25,001 to 100,000	100,001 or more	
Northeast	71.6	67.5	66.3	60.1	57.7	57.4	50.5	75.6	67.8
Southeast	69.1	67.0	65.6	60.3	59.0	58.4	57.7	55.6	63.2
Southwest	65.3	68.6	63.4	59.9	62.6	58.6	55.7	57.2	61.4
Middle States	61.8	56.6	51.2	48.6	47.0	45.0	48.7	47.3	51.4
Northwest	68.0	57.6	51.6	53.5	53.3	54.2	51.8	56.6	59.3
Far West	66.7	59.3	56.6	57.1	51.6	55.5	49.4	57.1	59.7
Northeast and Middle States	62.8	57.4	52.8	49.6	47.6	46.1	48.8	48.8	52.5
Southeast	69.1	67.0	65.6	60.3	59.0	58.4	57.7	55.6	63.2
Northwest, Southwest, and Far West	66.5	60.8	57.6	57.2	58.3	56.2	54.0	57.1	60.4
31 States	65.6	59.5	56.4	53.9	53.1	51.4	51.5	51.5	56.7

[1] The 31 States for which data were available were: Northeast—Maryland, New Hampshire, and Vermont; Southeast—Alabama, Florida, Kentucky, Louisiana, Virginia, and West Virginia; Southwest—Arizona, New Mexico, Oklahoma, and Texas; Middle States—Illinois, Indiana, Iowa, Minnesota, Missouri, Ohio, and Wisconsin; Northwest—Idaho, Kansas, Montana, Nebraska, North Dakota, South Dakota, Utah, and Wyoming; Far West—Nevada, Oregon, and Washington.

more, Maryland, is the only city reported in that group and 75.6 percent of the travel by its residents was for business purposes. In New York, cities having more than 100,000 population reported 57.5 percent of their travel for business purposes. These are higher percentages of travel for business than are reported in the cities in the lower population groups. The same condition may be noted in the Northwest and Far West regions and to a lesser degree in the Southwest region.

Table 6 presents data showing the estimated percentages of all motor-vehicle travel for business purposes in all States in 1940. These approximations have been determined on the following basis. The total vehicle-miles of travel by passenger cars and trucks in 1940 was determined for the several States on the basis of data from the traffic and road-use surveys. These mileages are preliminary and subject to revision but are believed to be approximately correct. The distribution of passenger-car travel by purpose in the 35 States where such data were available was used in estimating the distribution in States where there were no data. All truck travel was classified as travel for business purposes. The sum of truck travel and of passenger-car travel for business purposes was taken to be the amount of total motor-vehicle travel for business purposes. It will be seen that the percentage of business travel varies from 55.8 percent in Minnesota to 80.8 percent in Arkansas but that in 31 States and the District of Columbia the range is between 62 and 72 percent.

In 16 States, the classification of travel was made on the basis of business, social, and recreational purposes rather than the break-down into business and pleasure purposes. For those 16 States it is therefore possible to note the classification of travel otherwise considered as pleasure travel into the two purposes—social and recreational. It was found difficult to establish arbitrary definitions of what is business or social or recreational travel since so frequently a single trip accomplishes several purposes. The farmer, for example,

Table 6.—*Classification of estimated motor-vehicle travel by purposes in 1940*

State	Total travel for all purposes	Passenger car travel for—		Truck and bus travel	Total travel for business purposes	
		Social and recreational purposes	Business purposes		Amount	Percentage of total travel
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	
Alabama	3, 193	723	1, 601	869	2, 470	77. 4
Arizona	1, 219	380	549	290	839	68. 8
Arkansas	2, 323	446	1, 206	671	1, 877	80. 8
California	22, 558	7, 816	11, 725	3, 017	14, 742	65. 4
Colorado	2, 901	892	1, 492	517	2, 009	69. 3
Connecticut	5, 123	1, 949	2, 461	713	3, 174	62. 0
Delaware	787	162	477	148	625	79. 4
Florida	5, 230	1, 793	2, 414	1, 023	3, 437	65. 7
Georgia	5, 283	1, 505	2, 796	982	3, 778	71. 5
Idaho	1, 199	296	597	306	903	75. 3
Illinois	20, 115	8, 744	8, 102	3, 269	11, 371	56. 5
Indiana	7, 458	2, 615	3, 171	1, 672	4, 843	64. 9
Iowa	5, 814	2, 055	2, 814	945	3, 759	64. 7
Kansas	4, 962	1, 795	2, 304	863	3, 167	63. 8
Kentucky	4, 167	1, 120	2, 044	1, 003	3, 047	73. 1
Louisiana	3, 434	794	1, 861	779	2, 640	76. 9
Maine	1, 798	509	857	432	1, 289	71. 7
Maryland	4, 055	938	2, 343	774	3, 117	76. 9
Massachusetts	10, 598	3, 898	4, 922	1, 778	6, 700	63. 2
Michigan	14, 785	5, 871	6, 727	2, 187	8, 914	60. 3
Minnesota	6, 771	2, 991	2, 527	1, 253	3, 780	55. 8
Mississippi	2, 488	525	1, 226	737	1, 963	78. 9
Missouri	8, 268	2, 418	4, 136	1, 714	5, 850	70. 8
Montana	1, 425	408	694	323	1, 017	71. 4
Nebraska	3, 374	981	1, 620	773	2, 393	70. 9
Nevada	366	97	187	82	269	73. 5
New Hampshire	996	319	468	209	677	68. 0
New Jersey	12, 243	4, 470	5, 643	2, 130	7, 773	63. 5
New Mexico	1, 193	403	504	286	790	66. 2
New York	26, 086	9, 371	11, 830	4, 885	16, 715	64. 1
North Carolina	5, 972	1, 675	3, 111	1, 186	4, 297	72. 0
North Dakota	1, 152	348	577	227	804	69. 8
Ohio	16, 420	7, 040	6, 873	2, 507	9, 380	57. 1
Oklahoma	5, 240	1, 265	3, 068	907	3, 975	75. 9
Oregon	2, 957	1, 112	1, 381	464	1, 845	62. 4
Pennsylvania	19, 841	7, 434	9, 384	3, 023	12, 407	62. 5
Rhode Island	1, 948	747	942	259	1, 201	61. 7
South Carolina	2, 983	864	1, 605	514	2, 119	71. 0
South Dakota	1, 433	460	608	365	973	67. 9
Tennessee	5, 451	1, 415	2, 833	1, 203	4, 036	74. 0
Texas	17, 056	5, 103	7, 252	4, 701	11, 953	70. 1
Utah	1, 483	536	657	290	947	63. 9
Vermont	772	223	369	180	549	71. 1
Virginia	4, 602	1, 310	2, 330	962	3, 292	71. 5
Washington	4, 445	1, 413	2, 314	718	3, 032	68. 2
West Virginia	3, 202	1, 073	1, 354	775	2, 129	66. 5
Wisconsin	6, 972	2, 913	2, 866	1, 193	4, 059	58. 2
Wyoming	708	237	306	165	471	66. 5
District of Columbia	1, 478	441	844	193	1, 037	70. 2
Total	294, 327	101, 893	137, 972	54, 462	192, 434	65. 4

goes to town to market some of his goods, to obtain provisions, to see a movie, and to visit with other farmers and their families of that area. Similarly, the city wife may take her husband to work, her children to school, and do her marketing on the way home, all on the same trip. In the case of such overlapping purposes, it was generally the policy to classify the trip according to its major purpose.

Although this difficulty in classifying travel definitely according to purposes existed to a limited extent when the classification was made between business and pleasure purposes, the further classification of pleasure travel into social and recreational purposes was frequently even more difficult. A Sunday ride which included a visit to friends, a week-end or vacation trip which included a visit with relatives as well as the pursuit of purely recreational activities, a trip to the theater or to a dance which combined social and recreational functions were all difficult to classify definitely. However, a large majority of the trips could be classified correctly and the results in 16 States where this was done are shown in table 7.

The data in table 7 show that travel for recreational purposes is generally greater than that for social purposes. There appears to be no general pattern of travel in the several States, or among the several population groups. In some States recreational travel is about the same percentage of total travel as is travel for social purposes; in other States recreational travel is more than twice as great as social travel; in only four States is social travel greater than recreational travel.

These data suggest that there may actually be considerable differences in the purpose of travel in the several States depending on economic conditions and also on available recreational facilities in the State or adjacent States. It is also possible that the difficulty in distinguishing accurately between social and recreational travel may be the cause of the conditions observed in table 7. If this be true then the data in table 7 can be considered only as indicative of general conditions.

Certain other studies have been made recently which furnish additional information on the subject of the purpose of travel.[5] The Automobile Manufacturers Association made a special study in six States [6] of the road-use survey data previously collected in connection with the regular highway planning surveys. The special six-State study was particularly concerned with a detailed analysis of business or necessity use of passenger cars, and included as business use travel incurred taking children to and from school, a type of travel generally classed as nonbusiness travel in most of the State surveys.

Special attention was directed to business travel to determine the number of trips and amount of travel annually for each of the business purposes specifically classified in the road-use studies. It was the objective of this special study to determine for the various occupational groups among car owners how much of the business travel was concerned with transportation to and from work, how much with the actual conduct of business, how much for shopping and marketing, how much for taking children to and from school, etc.

BUSINESS TRIPS RELATIVELY SHORT BUT VERY NUMEROUS

Date obtained in these special studies indicate that while the percentage of travel for business purposes is high, the percentage of the total trips for business purposes is even higher. Table 8 shows that in four of the six States this condition was true. Complete data on the analysis of travel in Connecticut and Georgia are not yet available, the special analyses in those two States having been made by the Automobile Manufacturers Association personnel prior to the completion of the regular planning survey study. This relation is that which would be expected because today the automobile has come to occupy such an important part in everyday business activities. The large mileage that is accounted for in social and recreational purposes is generally accomplished in a smaller number of long trips such as vacation, fishing, and hunting trips and visits to friends. Business trips, though more numerous, are generally shorter—between home and place of employment—and it has not been generally realized what a large

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[5] See particularly, A Factual Survey of Automobile Usage, Automobile Manufacturers Association, 1941.

[6] Connecticut, Georgia, Indiana, Michigan, Nebraska, and Oregon.

TABLE 7.—*Classification of the total travel of passenger cars of 17 States for social and recreational purposes*

State	Distribution of travel by residents of—																	
	Unincorporated areas		Incorporated places having a population of—														All places	
			1,000 or less		1,001 to 2,500		2,501 to 5,000		5,001 to 10,000		10,001 to 25,000		25,001 to 100,000		100,001 or more			
	Travel for—		Travel for—		Travel for—		Travel for—		Travel for—		Travel for—		Travel for—		Travel for—		Travel for—	
	Social	Recreation	Social	Recreation	Social	Recreation	Social	Recreation	Social	Recreation	Social	Recreation	Social	Recreation	Social	Recreation	Social	Recreation
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Alabama	15.5	11.9	14.6	12.8	13.9	15.6	16.4	19.4	15.6	20.5	14.4	21.3	10.9	21.5	13.7	23.2	14.6	16.5
Arizona	13.5	23.9	14.4	26.5	18.9	24.5	15.3	26.1	16.9	26.6	------	------	12.0	32.4	------	------	13.8	27.1
Florida	15.8	16.0	11.6	18.0	16.0	21.7	11.9	30.9	13.2	28.8	12.5	39.5	14.7	36.7	10.0	38.0	12.9	29.7
Indiana	22.3	12.4	19.6	15.3	26.1	19.3	30.4	17.7	30.8	22.9	22.1	25.1	33.6	20.1	29.3	23.9	26.5	18.7
Kentucky	24.1	9.7	21.8	10.1	18.7	15.5	18.1	19.7	17.3	18.6	15.1	17.0	18.6	21.6	18.6	20.1	21.2	14.2
Louisiana	10.2	9.0	13.2	15.6	7.9	15.9	14.8	21.0	10.6	24.6	6.4	23.0	9.0	20.9	5.7	36.4	9.1	20.8
Montana	9.3	18.0	11.5	37.1	14.2	31.7	4.5	34.3	3.0	43.5	8.0	36.3	13.4	35.3	------	------	9.5	27.5
Nebraska	19.8	10.8	15.7	22.1	19.4	24.7	15.6	29.3	11.8	30.7	14.9	27.1	16.4	28.9	9.7	29.7	16.1	21.6
Nevada	5.9	21.5	10.9	37.3	3.1	26.4	4.6	34.2	2.5	29.6	6.2	38.7	------	------	------	------	5.3	28.8
New Hampshire	------	------	18.4	11.0	15.8	17.2	14.6	23.6	14.6	23.6	11.2	39.2	12.0	39.7	------	------	14.4	26.1
New Mexico	9.9	34.5	10.2	27.8	5.7	30.4	11.0	47.4	8.8	32.6	6.2	41.6	9.6	34.8	------	------	9.1	35.3
North Dakota	20.0	12.2	17.2	25.4	14.2	25.8	3.8	42.0	8.8	30.7	15.6	32.1	7.2	41.3	------	------	16.9	20.7
Oregon	17.5	18.2	18.8	25.4	22.3	30.9	14.7	31.8	19.7	34.6	16.5	32.1	28.0	34.7	16.4	31.2	17.7	26.9
South Dakota	20.4	14.9	14.9	30.0	13.2	37.7	12.7	33.1	10.6	40.1	11.1	36.3	18.3	33.7	------	------	16.1	27.0
Texas	25.4	12.2	18.2	16.7	19.6	20.6	18.3	22.0	20.0	23.4	19.3	26.4	19.3	25.8	19.5	24.5	21.4	19.9
Vermont	16.8	15.7	18.2	14.3	13.7	21.5	16.8	29.6	16.8	29.6	18.3	26.6	------	------	------	------	16.8	20.9
West Virginia	23.0	14.7	23.0	17.5	22.8	24.3	25.9	15.9	25.8	24.3	30.6	24.8	26.4	23.4	------	------	24.8	19.4
Total	20.2	13.5	17.1	19.5	18.1	21.9	17.7	24.1	17.9	26.5	16.4	28.8	19.3	26.6	17.5	27.8	18.6	21.5

mileage is accounted for by these relatively short but very numerous trips.

TABLE 8.—*Relation of business travel to total travel by passenger cars in four States with a comparison of number of trips and mileage traveled*

State	Percentage for business purposes of—	
	Total number of trips	Total annual vehicle-miles of travel
Indiana	76.3	54.8
Michigan	73.9	53.4
Nebraska	78.4	62.3
Oregon	80.9	55.4

Another important set of data obtained by the Automobile Manufacturers Association in their special study in the six States indicated the relation between the age of the vehicle and the percentage of use for business purposes. A previous study[7] by the Public Roads Administration in which an analysis of planning survey data was made reported the observed decrease in average annual travel of passenger cars in relation to their age. A similar study was made of the data in the six States and the results are shown in table 9. These data have not been adjusted in the manner followed by the several States in handling their respective motor-vehicle and road-use survey data but it is believed that, despite their having been taken directly from the road-use interviews, they satisfactorily indicate the general trends that are present.

It will be seen that the annual travel is less for the older vehicles, but that the older vehicles are used to a greater extent for business travel than are the newer vehicles. This is particularly true of the vehicles 9 years or more in age and indicates that these vehicles serve a more utilitarian purpose than do the newer cars. It also suggests that the older cars are not used as extensively for long trips away from home, because for the older vehicles the percentage of trips for business purposes is only slightly higher than in the case of the newer vehicles although the percentage of annual miles traveled for business purposes is considerably higher than in the case of the newer vehicles. It should also be noted in this connection that the average trip length for the older cars is approximately a third less than the average trip for the newest cars.

TABLE 9.—*Effect of age of passenger car on travel characteristics*[1]

Age of car, years	Average annual travel	Average number of round trips	Average trip length	Percentage of total travel for business purposes	Percentage of total trips for business purposes
	Miles	*Number*	*Miles*	*Percent*	*Percent*
1	10,768	619	17.4	54.7	77.2
2	9,628	620	15.5	54.7	77.6
3	8,592	593	14.5	54.2	77.2
4	8,106	604	13.4	53.5	77.5
5	7,624	545	14.0	52.8	75.2
6	7,083	524	13.5	53.0	74.8
7	6,718	496	13.5	53.9	75.9
8	5,804	468	12.4	56.6	76.6
9 and more	4,770	417	11.4	61.2	78.8

[1] Unadjusted data for Connecticut, Georgia, Indiana, Michigan, Nebraska, and Oregon, analyzed by the Automobile Manufacturers Association.

A special study by the Opinion Research Corporation, also reported in Automobile Manufacturers Association study indicates that the percentage of total travel for business purposes in the 35 States as reported in this article may be quite conservative. The Opinion Research Corporation survey covered a period of 1 week in the early part of the winter of 1940. It would be expected that some bias would result from such a seasonal choice since cars would be used more for shopping and other similar purposes and less for recreational purposes than at certain other times of the year. Results of that study for the test week in December showed 37 percent of the total travel to have been for business, 18 percent going to and from work, 11 percent for shopping, 2 percent taking children to and from school, and 32 percent for social and recreational purposes. It will be noted that the first three purposes, which are classed in the States' road-use surveys as business travel, account for 66 percent of the total travel while the results reported in this article for 31 States, in table 4, show 56.7 percent of the total travel for business purposes.

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[7] Distribution of Motor Vehicle Registrations and Tax Payments by Regions and Population Groups, by Ralph S. Lewis and Homer L. Baker, PUBLIC ROADS, January 1942.

It would be desirable, if adequate data were available, that comparisons be made between the purposes of travel today, and those in earlier years. Some data for early traffic surveys are available on this subject but are not comparable with the present data. However, the data for the early years are of interest in showing, for such observations as were made, the relationships existing in those years between the various purposes of travel. Although it is believed that the available data for various reasons represent a considerably smaller percentage of the travel for business purposes than actually existed it is significant that even under these conditions the percentage of travel for business purposes was high.

DATA FROM EARLY TRAFFIC STUDIES PRESENTED

A Connecticut transportation study made in 1923 classified all trip mileage according to business and nonbusiness travel and indicated that 44.5 percent of all passenger-car travel was for business purposes.[8] A definite relation was also observed in this survey between trip length and the purpose for which the car was used. For trips of less than 19 miles, 52.2 percent were for business while on trips of 500 miles or over, the percentage for business was only 12.6. In trips of 100 to 149 miles the percentage for business was 34.5 percent. In this travel study data were classified according to trips and not according to total mileage traveled, and the study was limited to the rural highways.

In the Pennsylvania survey conducted in 1924 52.8 percent of the passenger car traffic on the primary highway system of the State was found to be for business purposes.[9]

The survey made in the Cook County, Ill., area in 1924 revealed that at selected stations in that area 32.1 percent of the passenger car trips were for business purposes.[10]

The Ohio survey conducted in 1925 indicated that 55.4 percent of the passenger car traffic on the Ohio State Highway system was for business purposes.[11]

A Vermont survey in 1926 obtained data on the basis of which it was estimated that 33.7 percent of the travel on Federal-aid and numbered State-aid roads was for business purposes.[12]

A similar study in New Hampshire in the same year indicated that 24.6 percent of the passenger-car traffic on the trunk line system of the State was for business purposes.[13] This figure approximates that for Vermont and shows a considerably lower percentage for business travel on primary rural routes than in other States. This may be accounted for by the high volume of travel in those two States for recreational purposes by motor-vehicle owners of adjacent States. The higher percentage of travel for business purposes in Vermont as compared to New Hampshire may also be accounted for by the fact that at that time, and to a considerable extent even today, the tourist or recreational facilities of Vermont had not been developed or utilized as much as those in New Hampshire. Also the recreational facilities in the latter State were closer to the large populations of Connecticut, Massachusetts, and Rhode Island than were the existing facilities of Vermont.

In contrasting these figures with those from the current studies it should also be remembered that the recent figures include travel on all roads and streets and that the travel reported is that performed both in and out of the State by the motor-vehicle owners of that State. The earlier study essayed analysis of the travel as it occurred on any given system of roads regardless of that travel's in-State or out-of-State origin.

Several factors affect the comparability of the data from all these surveys. In all cases the figures from the earlier studies represent only traffic on rural roads and in several cases only that on the primary highway systems. There is thus omitted a large volume of traffic on city streets and on local rural roads where generally the percentage of travel for business purposes is considerably higher than on the main rural highways. In some of the surveys percentages of travel for business purposes were computed only on the basis of the number of trips observed which gives undue weight to the longer trips.

Data that are more nearly comparable with those of the present studies were obtained in financial-economic studies conducted in Wisconsin in 1930 and in Minnesota in 1933. Road-use surveys were integral parts of these studies and provided the following data relative to the purpose of travel of Wisconsin passenger-car owners.[14]

Place of ownership:	*Percentage of total travel for business purposes*
Rural areas	64.3
Incorporated places having a population of—	
2,500 or less	54.9
2,501 to 15,000	54.9
15,001 to 75,000	56.7
Milwaukee	62.9
State	60.1

These figures are somewhat higher than the planning survey figures obtained 6 years later where only 49.6 percent of the travel of residents of the State was found to be for business purposes. It is possible that there has been an increased use of the automobile for pleasure purposes in more recent years, especially since the economic depression of the early 1930's. This theory appears to be substantiated by the results of the Minnesota study conducted in 1933.[15] Analyses of the

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[8] The Connecticut Transportation Survey, PUBLIC ROADS, August 1926.

[9] Report of a Survey of Transportation on the State Highways of Pennsylvania, United States Bureau of Public Roads and the Pennsylvania Department of Highways, 1928.

[10] Report of a study of Highway Traffic and the Highway System of Cook County, Ill., by the United States Bureau of Public Roads and the Cook County Highway Department, 1925.

[11] Report of a Survey of Transportation on the State Highway System of Ohio, United States Bureau of Public Roads and the Ohio Department of Highways and Public Works, 1927.

[12] Digest of Vermont Highway Transportation Survey, by J. Gordon McKay, PUBLIC ROADS, December 1927.

[13] Report of a Survey of Transportation on the State Highways of New Hampshire, United States Bureau of Public Roads and the New Hampshire State Highway Department, 1927.

[14] The Wisconsin Financial Survey, PUBLIC ROADS, April 1933.

[15] Highways and Public Finance in Minnesota in 1932, by the United States Bureau of Public Roads and the Minnesota Department of Highways, 1934.

TABLE 10.—*Percentage of passenger cars traveling outside State of ownership*

State	Percentage of cars owned in—							
	Unincorporated areas	Incorporated places having a population of— 1,000 or less	1,001 to 2,500	2,501 to 10,000	10,001 to 25,000	25,001 to 100,000	100,001 or more	Total
Alabama	38.0	50.1	57.1	68.1	60.0	53.2	49.6	47.3
Arizona	31.5	34.2	37.3	48.0	------	42.7	------	38.2
Arkansas	22.9	24.3	39.4	34.7	36.4	32.6	------	27.6
Colorado	28.1	43.4	50.0	43.4	63.8	38.1	35.3	38.3
Florida	27.7	27.2	40.9	33.8	37.7	38.0	32.1	32.7
Idaho	45.7	58.3	57.1	70.2	51.9	------	------	53.7
Illinois	26.1	38.2	46.4	55.2	50.7	61.1	75.6	56.6
Indiana	31.2	44.2	49.1	48.1	50.6	58.5	48.7	43.4
Iowa	28.8	40.3	49.6	50.6	52.4	61.8	52.1	43.9
Kansas	34.3	50.4	54.4	63.0	68.6	49.1	75.0	51.5
Kentucky	34.2	38.3	54.8	62.0	57.8	69.0	51.2	44.9
Louisiana	21.2	27.0	31.9	32.0	40.4	45.7	36.9	31.6
Maryland	73.2	87.1	90.7	87.0	84.8	92.0	81.2	79.3
Michigan	24.0	27.2	31.7	32.5	49.4	36.5	37.6	34.7
Minnesota	21.0	32.8	31.1	39.2	38.6	------	37.6	31.2
Missouri	17.7	30.8	30.8	26.8	27.8	31.1	33.3	27.3
Montana	27.9	52.6	50.0	52.8	52.8	24.2	------	39.5
Nebraska	27.5	49.0	52.8	58.3	44.0	41.4	42.8	39.3
Nevada	55.2	20.2	58.4	72.8	83.8	------	------	66.6
New Hampshire	------	56.7	62.0	75.8	82.2	71.3	------	69.6
New Mexico	28.0	74.0	58.9	53.6	45.4	31.9	------	40.7
New York	34.5	39.2	39.2	42.0	48.1	47.2	47.0	44.6
North Dakota	24.9	40.8	65.2	51.6	63.3	75.3	------	37.0
Ohio	24.0	37.2	38.6	43.8	47.1	49.1	53.5	42.0
Oklahoma	30.8	30.8	40.6	50.9	53.6	34.2	56.9	42.6
Oregon	31.4	51.1	49.3	50.3	68.4	36.7	60.1	47.1
Pennsylvania	32.0	43.0	48.0	50.5	51.7	59.1	69.3	50.9
South Dakota	38.8	52.6	66.7	66.6	54.3	83.7	------	50.5
Texas	14.2	17.6	18.3	26.4	29.0	24.7	21.1	19.9
Utah	31.5	43.9	46.4	43.4	45.5	69.8	63.7	47.7
Vermont	45.2	53.8	52.8	61.5	59.8	------	------	51.6
Virginia	43.3	71.3	61.7	55.8	75.9	47.9	61.8	50.8
Washington	34.7	43.2	45.3	51.0	62.4	49.9	48.5	44.6
West Virginia	45.4	59.0	64.4	64.6	78.0	68.0	------	57.9
Wisconsin	23.3	37.1	49.9	42.5	53.9	50.4	51.2	40.4
Wyoming	57.1	63.7	64.8	67.2	60.8	------	------	61.2
Total	29.9	40.3	44.2	46.7	51.1	50.4	50.1	42.8

Minnesota data indicated that the following percentages of the total passenger-car travel of Minnesota motor-vehicle owners were for business purposes.

Place of ownership:	*Percentage of total travel for business purposes*
Rural areas	59.0
Incorporated places having a population of—	
2,500 or less	48.9
2,501 to 15,000	50.0
15,001 to 75,000	53.0
75,001 to 400,000	48.2
Minneapolis	51.9
State	52.7

In the road-use survey conducted as a part of the Minnesota planning survey three years later only 45.8 percent of the total travel was found to have been for business purposes, or, as in the case of Wisconsin, a definite decrease from the earlier figure. In Wisconsin, where the decrease was somewhat larger the period of time involved was twice as long as that between the two Minnesota studies and may account for the difference if the trend observed is actual. It should be noted, however, that Minnesota and Wisconsin, along with Ohio and Illinois are the only States included in this study in which less than 50 percent of the passenger-car travel was reported to be for business purposes. It is, therefore, possible that the trends noted in Minnesota and Wisconsin, of an increasing percentage of the total travel for social and recreational purposes, may not be true of other States where different economic and other conditions have resulted in higher percentages of travel for business purposes.

TABLE 11.—*Percentage of trucks traveling outside State of ownership*

State	Percentage of trucks owned in—							
	Unincorporated areas	Incorporated places having a population of— 1,000 or less	1,001 to 2,500	2,501 to 10,000	10,001 to 25,000	25,001 to 100,000	100,001 or more	Total
Alabama	36.7	35.1	41.7	36.3	22.7	21.4	12.9	32.1
Arizona	11.0	11.3	9.5	18.3	----	19.9	---	14.8
Arkansas	19.4	23.5	17.5	17.8	20.9	7.6	---	18.8
Colorado	11.1	13.0	12.0	5.3	22.1	4.4	15.6	11.8
Florida	16.0	8.9	13.2	4.7	6.1	4.9	5.1	9.1
Idaho	30.9	45.7	15.5	41.5	21.3		-	30.7
Illinois	13.8	17.0	14.9	8.6	3.2	10.7	6.1	10.1
Indiana	17.9	23.1	21.4	14.2	12.7	18.8	10.8	16.5
Iowa	15.1	15.5	16.6	13.2	9.2	7.4	3.4	11.9
Kansas	16.3	13.0	15.4	14.8	15.0	4.6	29.1	16.3
Kentucky	26.7	41.3	29.0	27.2	19.1	31.8	11.7	25.6
Louisiana	7.9	6.9	9.5	7.4	18.7	22.1	9.0	9.5
Maryland	43.5	59.2	64.8	44.6	29.5	66.8	43.1	45.2
Michigan	10.3	9.7	9.8	3.9	16.0	3.7	43.9	6.0
Minnesota	9.5	10.4	6.5	8.2	5.2	----	2.8	7.6
Missouri	18.6	27.6	27.6	12.4	10.7	14.1	11.8	16.5
Montana	14.4	14.5	18.5	15.5	15.2	1.1	--	13.8
Nebraska	15.2	20.8	18.4	18.4	9.1	7.7	11.0	15.4
Nevada	28.6	---	12.1	12.9	17.6	-	--	20.6
New Hampshire	---	29.2	32.2	43.8	41.8	24.0	----	33.8
New Mexico	7.2	17.5	23.9	19.3	21.8	16.9	--	13.7
New York	8.4	9.3	9.3	8.4	9.1	9.4	7.9	8.8
North Dakota	7.6	17.0	24.1	15.2	9.6	37.5	---	11.6
Ohio	9.5	13.3	10.5	12.6	15.5	9.8	11.5	11.5
Oklahoma	14.3	14.3	12.6	15.5	14.0	9.1	5.1	14.0
Oregon	16.2	17.6	23.2	10.6	32.6	5.1	24.5	18.3
Pennsylvania	14.5	16.1	17.8	12.6	10.8	19.1	19.7	16.4
South Dakota	25.6	25.8	33.7	12.9	12.6	26.1	-	24.1
Texas	6.0	7.9	6.6	9.8	12.4	11.8	8.8	8.1
Utah	18.5	23.8	23.8	17.1	7.4	32.4	18.1	20.0
Vermont	23.6	9.1	15.6	28.8	17.8	---	--	22.0
Virginia	30.6	41.8	40.0	20.8	37.8	11.1	.8	23.6
Washington	20.8	26.4	16.0	21.2	22.4	9.0	12.6	18.9
West Virginia	31.8	43.7	34.0	32.7	36.8	22.5	--	30.6
Wisconsin	5.6	9.4	11.3	7.0	14.6	9.4	2.4	7.0
Wyoming	32.2	32.2	19.8	30.0	14.7			28.0
Total	15.7	18.0	16.7	13.6	14.7	13.2	12.9	14.0

FORTY-THREE PERCENT OF PASSENGER CARS PERFORM SOME TRAVEL OUTSIDE STATE OF OWNERSHIP

From the data obtained in the road-use surveys it is possible also to make certain other analyses regarding travel characteristics. These items include (1) percentage of vehicles traveling in other States than that where owned;[16] (2) percentage of the total travel that is performed outside the State where the motor vehicle is owned; and (3) percentage of vehicles traveling in counties other than that where owned. Because of variations in size of counties, size of States, geographical conditions, location of marketing centers and location of population areas within the respective States, these figures cannot be used without proper qualifications. They are, however, informative of general characteristics.

Tables 10 and 11 show for each of the various population groups of 36 States the percentage of all passenger cars and of all trucks that travel outside the State at some time during the year. Other analyses of the road-use survey data have indicated that a large percentage of the total travel is accounted for by numerous short trips within a short radius of the place of residence. It is somewhat surprising, however, to discover in an era of surfaced roads connecting all parts of the country that almost 60 percent of the passenger cars, in the 36

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[16] Throughout this report, the term "place where owned" is used to indicate the place where the vehicle is regularly garaged.

States studied, never leave the State where they are owned in the course of their annual travel.

Table 10 shows that the lowest percentage of vehicles traveling outside the State occurs in the unincorporated areas and that generally the percentage traveling out of the State tends to increase as the size of the population group increases. There are few exceptions to this general tendency and most of those can be readily explained.

In Alabama, a somewhat smaller percentage of Birmingham (largest population group) cars travel outside the State than is the case for other population groups. This is undoubtedly caused by the location of that city in the interior of the State. A similar condition is evidenced in Colorado for cars owned by Denver residents. Denver, too, is toward the interior of the State and a large share of the vacation or holiday travel of Denver residents would be in the mountain areas to the west of the city.

In Florida a similar condition, to a somewhat lesser degree, is noted in the largest population group, which includes Jacksonville, Miami, and Tampa. Although Jacksonville is near the northern boundary of the State, Tampa and Miami residents have to travel 200 to 400 miles to get outside the State. As in the case of Alabama and Colorado, it is therefore understandable that a smaller percentage of vehicles in this population group should go outside the State, even though the proximity of Jacksonville provides a somewhat counterbalancing effect.

Similar conditions can also be observed in varying degrees in the 100,001 or more population group in Iowa where Des Moines is in the interior of the State, in Indiana where Indianapolis is in the center of the State though four other smaller cities in the group are nearer the State boundaries, and in Louisiana where New Orleans is some distance away from State boundaries.

The situation in Kentucky where Louisville is the only city in that group may be explained by the fact that although this city is on a State boundary the toll bridges across the Ohio River probably form an effective barrier to the north and disperse more of the traffic in other directions than would otherwise be the case.

It should be observed that in four Southern States—Arkansas, Florida, Louisiana, and Texas—the percentage of vehicles traveling outside the State is low. General economic conditions, including lower average cash incomes than elsewhere in the country, may account for this travel characteristic in some instances, but for Texas the distances which must be traveled from most population concentrations in order to reach a State boundary no doubt provides the explanation.

Although a low percentage of vehicles traveling outside the State is found in Michigan and Minnesota it is believed that the explanation there is somewhat different. Michigan has only a relatively small portion of its boundaries contiguous with other States. For the greater part of its boundaries it has four of the Great Lakes. In addition it has varied and extensive natural and developed recreational facilities within its own boundaries so that its residents need not drive outside the State for their recreation. Conditions in Minnesota are somewhat similar. To the north lies a sparsely populated section of Canada, which serves somewhat as a natural barrier to out-of-State travel in that direction. States to the east, south, and west of Minnesota offer few recreational facilities that the State itself does not have. It might be thought that similar conditions would exist in Wisconsin but it must be remembered that the most densely populated areas are located in the southeastern part of the State which is adjacent to the Chicago metropolitan area, a fact which might be expected to create greater out-of-State travel by Wisconsin residents.

The rather high number of Maryland cars traveling out-of-State, as shown in table 10 is to be expected because of the size and shape of the State. The narrow western panhandle and the Baltimore concentration within 40 miles of the State boundary are both conducive to a large percentage of cars traveling outside the State. A similar condition exists in New Hampshire, a relative small State, where approximately 70 percent of the population is concentrated in the five southern counties which account for only about 40 percent of the area of the State.

The high percentage of Nevada cars traveling outside the State can likewise be accounted for by the distribution of population in the State. Approximately 39 percent of the population is located within a short distance of the California boundary in the Reno-Carson City area, 24 percent in the northeast area adjacent to Idaho and Utah and 9 percent in the southern tip of the State in the Las Vegas area.

FOURTEEN PERCENT OF TRUCKS PERFORM SOME TRAVEL OUTSIDE STATE OF OWNERSHIP

Table 11 presents similar data for trucks. It will be noted that the percentage of trucks traveling outside the State of ownership is relatively small compared to that for cars.

As in the case of cars, the 45.2 percent of Maryland trucks going outside the State is definitely higher than for any other State.

Only four other States reported more than 30 percent of their trucks traveling outside the State. These were Alabama, Idaho, New Hampshire, and West Virginia. This condition can be explained in the latter State by the heavy industrial development in the western part of the State and also in the panhandle region near the Pittsburgh area.

Tables 12 and 13 show for 34 States the percentages of cars and of trucks traveling outside the counties of ownership in the respective States. Adequate bases of comparison are even more lacking here than in the case of out-of-State travel. In addition to such factors as the degree of urbanization of the county, the location of shopping and marketing centers, and the condition of road systems in the county of residence and adjacent counties, the size of the county is probably the most important single item. Counties throughout the United States vary widely in size from Arlington County, Va., of 25 square miles to San Bernardino County, Calif., with 20,175 square miles.

Only 6 of the 34 States reported that less than 80 percent of the passenger cars traveled outside the county of residence during the year. Four of these were Western States—Arizona with an average county area of 8,129 square miles, Nevada with an average county area of 6,460 square miles, New Mexico with an average county area of 3,952 square miles and Wyoming with an average county area of 4,241 square miles.

A comparison of Nevada data in tables 10 and 12 reveals an apparent contradiction of table 10 by table 12. This is due to the nature of the out-of-county travel analysis. In table 10 Nevada was shown to have the third from the highest percentage of cars

TABLE 12.—*Percentage of passenger cars traveling outside county of ownership* [1]

State	Percentage of cars owned in—							
	Unincorporated areas	Incorporated places having a population of—						Total
		1,000 or less	1,001 to 2,500	2,501 to 10,000	10,001 to 25,000	25,001 to 100,000	100,001 or more	
Alabama	84.7	93.9	94.4	94.8	89.6	82.3	86.5	86.9
Arizona	61.9	61.6	77.9	63.2	------	77.6	------	66.8
Arkansas	71.2	84.2	90.1	90.1	91.3	91.0	------	80.0
Colorado	90.0	96.3	99.2	98.8	93.3	95.7	98.0	94.8
Florida	76.9	89.4	91.1	86.2	87.9	76.5	79.6	81.3
Idaho	85.0	94.0	95.7	94.7	95.8	------	------	89.9
Illinois	80.7	89.9	88.3	91.1	88.2	88.8	82.1	85.0
Indiana	78.5	89.3	92.7	92.4	90.1	84.3	75.5	81.9
Iowa	84.4	92.6	93.4	93.0	92.5	79.5	91.5	87.7
Kansas	83.6	92.6	94.5	91.1	93.1	91.2	86.6	87.7
Kentucky	83.4	92.2	94.0	92.9	89.6	87.3	76.3	83.2
Louisiana	75.6	91.7	91.9	85.8	79.0	70.6	55.5	70.9
Maryland	86.0	90.6	92.0	94.7	96.0	88.3	95.4	90.5
Michigan	86.1	93.4	94.6	90.4	90.8	95.1	90.8	90.8
Minnesota	80.7	95.8	94.5	96.4	97.5	------	98.3	91.6
Missouri	78.4	89.7	89.7	83.6	33.9	84.0	75.2	76.3
Montana	83.8	95.1	97.9	92.9	95.8	93.6	------	90.4
Nebraska	79.4	90.6	89.8	84.6	92.8	85.4	55.7	80.6
Nevada	54.6	62.3	91.1	54.0	87.8	------	------	63.2
New Mexico	67.1	77.0	84.7	68.8	81.8	87.1	------	74.1
New York	81.2	90.9	90.9	90.1	90.4	91.9	83.7	81.6
North Dakota	81.5	95.0	92.0	95.9	87.7	77.8	------	86.3
Ohio	85.8	92.7	94.8	93.2	93.4	94.0	90.4	90.4
Oklahoma	83.1	83.1	87.7	88.7	96.6	64.6	91.9	86.9
Oregon	88.0	92.7	95.8	98.1	95.6	98.1	92.9	91.9
Pennsylvania	80.4	99.7	83.5	91.9	92.8	92.4	80.1	86.5
South Dakota	73.7	83.7	95.2	94.1	91.2	98.4	------	85.1
Texas	83.8	93.1	94.8	93.0	90.6	86.5	82.1	86.2
Utah	92.6	93.6	89.6	95.0	84.9	99.3	97.6	94.3
Virginia	81.5	94.1	95.3	92.9	89.7	92.0	94.9	87.1
Washington	82.8	90.2	91.5	92.7	89.0	91.2	90.2	87.9
West Virginia	88.0	91.9	91.0	96.2	95.6	92.9	------	90.8
Wisconsin	86.1	94.4	93.9	95.4	97.0	92.0	96.6	92.2
Wyoming	74.0	83.0	88.5	84.4	80.2	------	------	79.5
Total	81.5	90.0	89.4	89.8	88.3	88.1	85.3	85.2

[1] Travel outside the county refers only to travel in other counties in the State of ownership. Data for New Hampshire and Vermont not analyzed on a county basis.

TABLE 13.—*Percentage of trucks traveling outside county of ownership* [1]

State	Percentage of trucks owned in—							
	Unincorporated areas	Incorporated places having a population of—						Total
		1,000 or less	1,001 to 2,500	2,501 to 10,000	10,001 to 25,000	25,001 to 100,000	100,001 or more	
Alabama	83.2	87.3	95.3	92.1	76.0	59.7	67.0	80.8
Arizona	32.3	45.0	44.6	20.7	------	31.2	------	31.7
Arkansas	64.8	71.8	64.9	65.0	38.7	44.3	------	61.6
Colorado	72.4	77.1	79.5	78.9	57.5	62.5	74.8	72.8
Florida	50.3	58.0	62.9	45.1	29.5	18.4	35.4	42.6
Idaho	64.8	79.0	68.9	70.2	79.6	------	------	69.1
Illinois	54.3	60.8	51.4	38.8	30.5	27.2	8.4	31.4
Indiana	53.9	65.7	57.9	50.5	37.2	33.3	22.3	44.0
Iowa	52.7	63.9	67.5	52.9	45.0	23.4	31.3	47.3
Kansas	56.5	62.0	51.6	37.5	42.7	31.4	43.8	47.7
Kentucky	72.0	86.5	68.7	69.1	41.2	47.9	18.1	61.4
Louisiana	56.6	57.7	61.0	61.0	56.1	40.9	48.6	40.4
Maryland	66.0	73.0	77.2	75.0	68.2	40.4	77.4	70.4
Michigan	55.5	59.7	63.5	37.8	47.2	52.3	31.2	40.8
Minnesota	56.7	68.3	64.2	53.7	53.2	------	45.6	54.1
Missouri	66.1	66.3	66.3	57.7	59.6	34.6	23.9	45.7
Montana	74.9	73.2	75.1	70.6	65.9	44.1	------	70.5
Nebraska	47.5	68.4	69.7	50.9	52.0	31.9	24.3	50.5
Nevada	37.7	16.2	51.4	32.9	24.0	------	------	30.0
New Mexico	39.3	56.3	44.6	25.7	38.9	37.2	------	36.9
New York	45.3	49.8	49.8	52.9	52.3	51.0	31.4	31.4
North Dakota	52.7	79.9	59.3	72.5	49.7	37.5	------	57.6
Ohio	63.7	71.6	71.0	58.2	54.2	47.7	34.5	50.7
Oklahoma	63.2	63.2	62.2	60.1	75.3	66.0	43.1	62.1
Oregon	62.7	65.2	60.8	61.5	52.9	86.4	43.6	58.8
Pennsylvania	53.6	92.2	29.9	52.8	53.1	59.2	36.1	49.1
South Dakota	51.7	54.1	64.2	52.5	49.1	59.1	------	53.9
Texas	73.4	80.7	83.4	77.1	68.3	64.3	59.5	69.8
Utah	79.8	84.5	80.6	76.0	74.1	74.3	55.7	74.0
Virginia	65.3	67.2	64.4	79.6	70.7	57.1	53.7	63.3
Washington	61.6	67.5	44.3	60.2	52.6	59.7	28.5	48.1
West Virginia	71.1	80.3	74.7	55.4	57.5	64.0	------	66.8
Wisconsin	48.6	65.3	51.7	53.1	54.9	35.0	18.5	46.3
Wyoming	51.2	46.7	44.3	37.5	33.6	------	------	44.8
Total	59.7	65.5	60.4	55.7	50.7	44.6	34.5	49.5

[1] Travel outside the county refers only to travel in other counties in the State of ownership. Data for New Hampshire and Vermont not analyzed on a county basis.

traveling outside the State. This is not true for travel outside the county as shown in table 12, nor is the percentage of cars traveling outside the counties of residence in Nevada as great as the percentage of cars traveling outside the State. Travel outside the county refers only to travel in other Nevada counties so that a passenger car might be driven from Washoe County into California but not driven into any other counties in Nevada during the year. The Nevada data, and those for most other States also, are therefore a measure of use of other road facilities in the State rather than entirely a measure of travel outside limited boundaries. It should be noted, however, that the Nevada State percentage is controlled in the case of table 12 by the travel of residents of unincorporated areas and those in places of 2,501 to 10,000 population because only in those two population groups are the percentages of those traveling outside the county of residence smaller than the percentages of those traveling outside Nevada.

Table 13 presents data on truck travel in other counties of the respective States. While the total for the 34 States shows that almost half the trucks traveled in other counties, as compared to 85.2 percent of the cars, the percentage exceeds 70 in Alabama, Colorado, Maryland, Montana, and Utah.

It is noted, however, that again the Western States of Arizona, Nevada, and New Mexico report relatively low percentages of their trucks traveling in other counties. Similar low percentages are reported in Illinois, Michigan, and New York, where the travel of large numbers of trucks is limited to the metropolitan areas of Cook County in which Chicago is located, Wayne County, with Detroit, and the five counties or boroughs of New York City, all of which were classed as one county in the New York study.

The somewhat erratic percentages of truck travel in other counties as compared with car travel are undoubtedly more closely allied with the factors of shopping and marketing center locations. Size of the county, imposing geographic limitations on car travel, undoubtedly affects truck travel also but the economic factors involved in the operation of commercial vehicles are undoubtedly more significant in the case of trucks than in the case of cars.

OUT-OF-STATE TRAVEL A LOW PERCENTAGE OF TOTAL

As a further development of the data in tables 10 and 11 an analysis was also made of the percentage of the total vehicle-miles traveled by cars and trucks that occurred outside the State where the vehicle was registered. This information is shown in tables 14 and 15.

Although 43 percent of all cars and 14 percent of all trucks, according to the data shown in tables 10 and 11, make at least one trip outside the State of ownership during the period of a year, tables 14 and 15 indicate that the percentage of the total travel so accounted for is very much less. This, of course, is consistent with other data obtained in the road-use surveys that show the small percentage of travel performed at any considerable distance from home.[17]

In the 36 States for which data are reported only 7.7 percent of all passenger-car travel and 5.5 percent of all truck travel was outside the respective State of ownership. The highest percentage of out-of-State travel for cars is found in New Mexico and Nevada, both Western States. High percentages of such travel are also found in the Eastern States of New Hampshire and Maryland whose State areas are restricted. Generally, with the exception of Washington and Oregon, out-of-State travel is a higher percentage of the total travel of car owners in most of the Western States than it is in the case of Eastern States. This can be observed in the case of Arizona, New Mexico, Nevada, Idaho, Utah, and Wyoming.

TABLE 14.—*Percentage of total travel of passenger cars outside the State of ownership*

State	Percentage of travel outside the State of cars owned in—							
	Unincorporated areas	Incorporated places having a population of—						Total
		1,000 or less	1, 001 to 2, 500	2, 501 to 10, 000	10, 001 to 25, 000	25, 001 to 100, 000	100, 001 or more	
Alabama	3.9	4.6	7.5	9.7	8.8	9.4	6.8	6.2
Arizona	13.4	14.9	12.9	13.5	------	15.3	------	14.0
Arkansas	13.9	11.8	11.8	13.4	12.4	12.6	------	13.1
Colorado	5.1	10.0	12.9	8.3	11.2	7.5	8.5	8.3
Florida	10.2	11.7	12.8	13.8	13.6	17.6	12.5	12.8
Idaho	7.8	12.0	13.5	17.7	10.3	------	------	11.4
Illinois	3.1	4.3	5.4	7.5	8.9	10.1	13.4	9.8
Indiana	3.8	5.3	5.2	6.7	8.7	10.8	7.7	6.6
Iowa	3.8	6.2	9.7	11.2	12.3	12.5	10.0	8.5
Kansas	6.6	9.1	12.3	16.6	13.3	7.9	17.6	11.6
Kentucky	5.3	4.1	9.6	9.5	10.4	15.7	8.8	7.9
Louisiana	6.3	4.6	9.1	8.4	9.9	16.4	14.1	10.4
Maryland	13.1	15.6	20.6	21.8	18.4	16.8	13.0	14.4
Michigan	2.0	2.9	3.5	4.5	7.4	4.7	5.8	5.0
Minnesota	2.3	4.5	4.2	5.9	4.9	------	5.1	4.3
Missouri	2.8	4.4	4.4	4.3	4.6	5.4	6.4	11.2
Montana	7.6	7.5	9.8	9.3	10.3	8.0	------	8.4
Nebraska	3.5	7.1	10.3	11.4	7.1	7.2	9.0	6.9
Nevada	13.9	11.7	21.8	17.3	15.5	------	------	15.9
New Hampshire	------	10.0	11.7	13.5	19.1	13.4	------	13.5
New Mexico	12.3	19.0	12.2	24.7	15.9	10.8	------	15.9
New York	3.9	4.8	4.8	7.1	9.0	9.0	9.3	8.0
North Dakota	4.4	9.0	15.2	10.7	16.1	29.5	------	9.1
Ohio	2.5	4.7	5.9	7.6	6.6	7.5	8.3	6.4
Oklahoma	5.8	5.9	6.4	6.2	6.9	6.9	9.4	6.9
Oregon	6.4	7.6	9.3	8.8	8.0	8.2	13.1	9.5
Pennsylvania	4.2	5.8	7.1	7.8	7.4	10.5	13.5	8.6
South Dakota	6.2	11.9	14.3	14.3	13.4	16.3	------	10.8
Texas	1.8	3.4	2.3	3.1	4.2	3.7	2.9	2.7
Utah	7.8	6.9	9.4	10.2	9.7	14.5	15.8	11.8
Vermont	7.2	9.5	4.9	12.2	8.0	------	------	8.1
Virginia	5.8	9.7	8.2	6.9	14.5	5.4	6.4	6.6
Washington	5.7	7.8	7.0	8.9	10.6	6.5	9.1	7.9
West Virginia	4.7	10.4	10.7	13.5	18.0	13.2	------	9.9
Wisconsin	2.8	4.5	6.6	4.3	8.5	7.8	6.2	5.6
Wyoming	10.5	10.6	14.8	14.3	15.5	------	------	12.9
Total	4.6	6.2	7.0	8.1	8.8	9.2	9.1	7.7

When the States are studied individually it is also found that in two-thirds of them the percentage of total passenger-car travel that is out-of-State is smallest for the unincorporated areas. The exceptions occur generally for the smaller places. Out-of-State travel for cars in the 1,000 or less population group is smaller than similar travel for cars in unincorporated areas in Arkansas, Kentucky, Louisiana, Montana, Nevada, and Utah. Similar conditions exist for the 1,001 to 2,500 population group for Arizona, Arkansas, New Mexico, and Vermont, for the 2,501 to 10,000 and 10,001 to 25,000 population groups only for Arkansas; for the 25,001 to 100,000 population group for Arkansas, New Mexico, and Virginia, and for the 100,001 or more population group only for Maryland.

On the basis of the percentage of trucks traveling outside the State annually (table 13) the data in table 15 appear reasonable. A considerably smaller percentage of trucks than of cars travels outside the State annually; likewise, a considerably smaller percentage of the total travel of trucks is outside the State than is the case for cars.

Trucks owned in unincorporated areas traveled outside the State less than those of other population groups. However, there is less variation among the population groups than in the case of cars. It is also noteworthy that in only eight of the States was the lowest percentage of out-of-State travel performed by trucks owned in unincorporated areas. This is entirely different from the tendency noted in car travel.

TABLE 15.—*Percentage of total travel of trucks outside the State of ownership*

State	Percentage of travel outside the State of trucks owned in—							
	Unincorporated areas	Incorporated places having a population of—						Total
		1,000 or less	1,001 to 2,500	2,501 to 10,000	10,001 to 25,000	25,001 to 100,000	100,001 or more	
Alabama	6.0	5.9	9.6	6.1	5.9	7.1	6.9	6.5
Arizona	7.0	2.7	2.9	7.1	------	29.2	------	16.1
Arkansas	15.8	19.5	11.4	8.9	17.8	27.9	------	16.3
Colorado	4.3	5.2	1.2	1.1	8.8	2.2	14.5	6.7
Florida	5.0	9.1	5.5	6.8	3.7	4.5	5.8	5.6
Idaho	7.2	48.0	10.3	12.1	20.8	------	------	16.9
Illinois	2.8	2.0	1.0	1.9	.4	2.7	7.1	4.0
Indiana	6.4	7.0	5.5	8.1	10.5	9.4	10.6	8.4
Iowa	3.0	10.7	3.7	5.1	6.8	7.1	.9	5.9
Kansas	3.9	3.0	7.5	10.4	3.9	.7	12.3	5.8
Kentucky	5.2	8.4	4.9	6.0	5.8	8.1	9.6	6.3
Louisiana	3.5	.9	3.7	1.6	5.8	15.8	5.0	4.9
Maryland	16.4	18.5	19.7	24.3	20.9	19.7	8.1	14.2
Michigan	2.1	2.1	3.4	3.5	4.9	2.4	3.6	3.2
Minnesota	1.2	2.7	3.2	4.1	2.1	------	3.5	2.5
Missouri	2.6	7.1	7.1	1.0	3.3	4.6	12.7	6.9
Montana	3.9	4.0	6.0	5.2	1.9	1.9	------	3.7
Nebraska	5.4	13.0	12.0	15.2	11.3	11.8	9.0	11.0
Nevada	14.0	------	37.8	14.9	2.8	------	------	16.4
New Hampshire	------	5.8	11.9	10.6	19.0	13.5	------	12.3
New Mexico	8.2	9.5	10.9	14.8	8.7	4.1	------	9.8
New York	4.1	2.5	2.5	3.3	2.4	2.8	6.8	5.0
North Dakota	1.9	2.6	3.5	4.2	4.5	18.5	------	4.2
Ohio	3.5	2.9	3.3	4.6	4.7	6.4	5.5	4.8
Oklahoma	3.0	3.0	3.3	2.6	3.4	4.7	1.9	2.8
Oregon	5.0	7.8	6.9	7.8	10.9	3.2	8.4	6.9
Pennsylvania	5.5	9.7	7.4	5.3	5.9	8.5	7.7	6.8
South Dakota	5.0	9.4	11.9	6.2	11.6	7.7	------	8.5
Texas	1.2	1.3	.8	1.5	1.8	2.3	1.1	1.4
Utah	6.7	6.4	7.0	6.3	7.3	7.9	9.1	7.4
Vermont	12.9	1.2	5.2	9.2	21.8	------	------	12.5
Virginia	1.3	2.7	9.3	.9	2.3	.8	.2	1.4
Washington	3.3	3.3	2.7	4.0	6.0	.2	3.7	3.6
West Virginia	4.6	2.5	9.2	9.4	10.9	10.5	------	7.7
Wisconsin	1.3	4.1	2.8	1.7	5.4	3.5	1.6	2.5
Wyoming	12.0	10.4	7.8	14.5	16.9	------	------	12.2
Total	4.3	6.6	4.6	5.0	5.6	6.3	6.2	5.5

In another respect the travel of cars and trucks is strikingly similar. In both cases, in 13 of the 36 States the highest percentage of out-of-State travel was reported for cars and trucks owned in the largest population group. In the case of cars in 10 other States the highest percentage for out-of-State travel was in the next to the largest population group; this condition held for trucks in 6 States.

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[17] Preliminary Results of Road Use Studies, by Robert H. Paddock and Roe P. Rodgers, PUBLIC ROADS, May 1939.

SUMMARY

The data presented in this report indicate generally certain travel characteristics of passenger car and truck owners in the several States.

A small percentage of the total travel of all trucks is for other than business or commercial purposes. For the States for which such data are available only 2.5 percent of all truck travel was for other than business or commercial purposes.

More than half, or 56.7 percent, of all passenger-car travel in 31 States was for business purposes, and this travel combined with the travel by trucks indicates that approximately 65 percent of all motor-vehicle travel is for business purposes.

A classification of pleasure travel into social and recreational travel indicates that, although the percentages vary from State to State, pleasure travel is divided approximately equally between social and recreational travel.

Although older cars travel fewer miles each year and make shorter trips, a higher percentage of their total travel is for business purposes than is the case for the newer cars.

Cars and trucks owned in unincorporated areas make a smaller percentage of trips outside the State where they are owned, are used to visit fewer other counties in the State of ownership, and perform a smaller percentage of their total travel outside the State of ownership than do the cars and trucks of any other population group.

In the 36 States studied 42.8 percent of the cars but only 14.0 percent of the trucks traveled outside the State of ownership during the year.

In 34 States, almost 15 percent of the cars and 50.5 percent of the trucks did not travel into other counties of the State of ownership during the year studied.

In 36 States included in the study, 7.7 percent of the total travel of the passenger cars and 5.5 percent of the total travel of trucks was performed outside the State of registration.

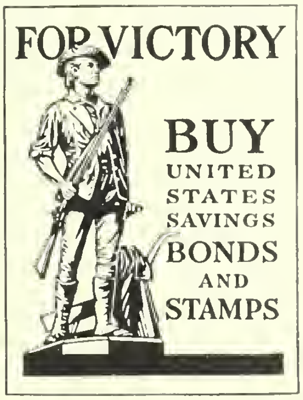
Generally, the extent of travel including the percentage of vehicles traveling outside the county and State of registration, and the percentage of total travel performed outside the State, increase as the size of the population group increases.

PUBLICATION REPORTS RESEARCH ON FLEXIBLE PIPE CULVERTS

The first rational analysis ever made of the behavior of flexible types of culvert pipe, based on accepted principles of mechanics, has recently been published as Bulletin No. 153 of the Iowa Engineering Experiment Station. This bulletin, The Structural Design of Flexible Pipe Culverts, by M. G. Spangler, reports the results of a joint investigation by the Public Roads Administration and the Iowa Engineering Experiment Station.

The study reveals that the thin-ring elastic analysis is valid for calculation of deflections of corrugated metal pipe. This analysis is used as a basis for deriving a design formula for pipe deflection which evaluates the effect of fill load, pipe size, bedding, properties of the soil, moment of inertia of the pipe wall, modulus of elasticity of the metal, and time of service.

Single copies of this 80-page bulletin may be obtained without charge from the Iowa Engineering Experiment Station, Ames, Iowa.



STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF MARCH 31, 1942

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$6,456,558	$3,207,060	228.2	$4,053,534	$2,134,580	115.2	$415,400	$206,050	0.3	$2,622,386
Arizona	1,683,133	1,181,967	68.1	1,138,071	835,693	40.5	384,302	283,319	8.4	1,993,612
Arkansas	3,634,317	1,665,856	57.4	1,173,174	585,470	60.1	152,496	98,396	.5	1,722,574
California	8,938,376	4,823,259	149.3	5,016,942	3,321,288	56.3	472,203	307,723	14.3	4,464,555
Colorado	2,297,094	1,278,286	154.6	3,618,770	2,255,799	155.6	1,165,565	652,450	41.1	2,191,480
Connecticut	1,419,554	696,181	17.1	1,715,156	821,787	18.4	481,485	239,471	9.4	1,228,870
Delaware	606,588	297,972	17.1	441,010	216,559	9.0	268,040	134,020	8.4	1,498,572
Florida	1,342,189	698,897	66.7	3,131,576	1,753,655	49.2	512,242	293,621	4.6	3,031,766
Georgia	2,994,295	1,495,854	111.4	7,399,882	3,709,691	281.9	2,914,444	1,457,222	122.2	7,032,315
Idaho	1,812,907	1,182,907	93.7	1,283,694	917,647	63.1	58,224	36,000	.1	1,982,623
Illinois	4,129,995	2,014,092	89.7	7,033,216	3,564,536	121.7	1,361,200	680,600	6.2	7,107,545
Indiana	4,621,229	2,127,735	75.1	6,572,139	3,207,304	92.8	800,300	400,150	14.3	2,898,930
Iowa	3,903,246	1,887,215	170.6	4,348,332	1,758,282	130.2	493,917	91,450	13.1	2,533,968
Kansas	4,678,783	2,375,589	263.2	6,127,089	3,223,190	253.4	997,444	442,350	47.9	4,804,799
Kentucky	4,063,448	2,002,298	146.9	6,150,960	2,932,510	118.8	2,004,624	935,397	15.1	2,022,821
Louisiana	1,514,168	801,291	33.5	1,382,924	914,401	28.5	2,553,917	1,251,767	56.3	4,265,087
Maine	948,788	469,405	26.8	2,070,392	1,061,696	27.8	78,610	39,305	.1	1,125,422
Maryland	2,765,267	1,381,529	29.6	3,446,909	1,466,154	15.0	35,000	17,500		1,587,207
Massachusetts	2,351,882	1,178,572	17.3	2,239,503	1,159,262	14.9	1,273,400	658,873	9.1	3,817,721
Michigan	8,467,297	4,215,312	174.2	2,919,748	1,891,824	50.6	976,500	561,825	9.5	3,295,032
Minnesota	4,463,072	2,183,470	378.0	9,522,463	4,717,659	388.9	360,358	180,179	19.6	3,516,916
Mississippi	5,332,567	2,602,918	283.3	3,744,324	1,874,762	218.1	21,400	13,700		2,306,444
Missouri	5,059,170	2,580,179	160.1	10,477,229	5,955,231	193.2	1,888,556	630,413	24.8	4,213,849
Montana	2,181,671	1,259,532	117.1	4,026,424	2,350,912	173.9	387,977	224,823	55.2	4,451,757
Nebraska	2,921,742	1,430,733	277.8	5,228,396	2,629,153	476.7	82,976	41,488	10.9	4,494,991
Nevada	2,233,884	1,910,826	110.6	742,339	643,031	22.0	297,496	257,831	3.4	1,401,980
New Hampshire	356,013	199,086	6.0	1,700,559	1,022,873	20.2				831,542
New Jersey	2,998,818	1,466,835	26.6	2,988,902	1,494,371	16.2				3,045,348
New Mexico	1,629,712	1,018,416	112.8	1,098,866	778,893	63.2				2,825,765
New York	9,599,156	4,820,744	126.7	7,932,755	5,062,640	80.6	1,035,400	620,300	12.5	5,186,477
North Carolina	4,004,928	2,112,295	162.8	2,918,474	1,596,539	123.5	575,070	287,535	8.8	3,663,261
North Dakota	3,221,529	1,882,679	287.6	2,566,134	1,451,752	203.9	2,448,925	1,279,699	205.5	4,006,811
Ohio	10,008,288	5,303,101	97.9	11,204,292	6,139,043	75.3	5,385,210	2,201,043	28.0	3,212,120
Oklahoma	2,505,159	1,267,267	113.8	2,530,422	1,353,270	51.6	1,694,180	896,004	60.2	6,252,003
Oregon	2,783,897	1,644,626	73.0	3,200,754	1,659,318	71.4	662,730	503,480	36.2	1,408,320
Pennsylvania	10,068,733	4,993,380	98.5	10,310,764	5,304,962	78.6	1,442,004	902,291	10.8	4,393,457
Rhode Island	1,196,895	596,429	10.0	644,776	322,368	4.8	644,448	322,224	2.0	945,132
South Carolina	2,190,404	1,063,179	92.1	3,977,759	2,494,452	93.7	714,490	430,100	18.3	1,467,885
South Dakota	2,232,437	1,315,959	278.6	4,864,023	3,202,055	542.5	545,110	325,580	59.3	2,880,650
Tennessee	4,045,617	2,167,783	106.5	5,060,450	2,999,979	103.1	250,044	142,648	3.5	3,481,260
Texas	11,542,429	5,636,837	570.5	11,390,330	5,419,551	365.6	1,882,230	731,310	64.2	9,529,623
Utah	1,206,046	907,446	53.8	1,885,271	1,428,236	46.7				1,253,842
Vermont	819,207	448,606	28.8	1,195,762	716,441	20.7	36,906	18,453	.3	392,521
Virginia	3,941,867	1,839,199	74.7	3,510,798	1,737,536	57.3	35,490	17,745	.4	2,891,214
Washington	1,685,728	897,956	25.7	2,605,052	1,408,897	31.5	43,686	23,400	1.0	2,207,228
West Virginia	2,766,115	1,372,978	51.4	2,482,738	1,231,611	29.3	338,900	166,750	1.8	2,206,248
Wisconsin	2,205,117	1,114,356	92.7	5,618,496	3,567,693	163.9				4,282,329
Wyoming	1,404,738	964,312	148.3	1,807,737	1,339,848	123.6				1,313,397
District of Columbia	1,072,296	547,140	3.5	227,782	135,272	.9	772,850	479,688	.3	217,454
Hawaii	133,296	66,648	2.8	1,078,243	792,916	10.9	378,475	359,003	4.3	1,574,165
Puerto Rico	493,285	238,835	3.9	2,138,655	1,056,215	18.0	415,525	255,509	3.5	606,323
TOTALS	174,932,930	90,835,027	5,966.4	199,943,990	109,618,807	5,572.8	39,739,749	20,098,685	1,015.7	151,687,597

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF MARCH 31, 1942

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$1,392,972	$695,108	62.5	$519,982	$285,130	25.2	$206,701	$96,830	6.0	$581,490
Arizona	245,110	174,247	14.1	15,183	13,767		129,775	63,749	6.8	546,644
Arkansas	623,246	237,499	33.1	483,854	241,861	26.5				335,559
California	1,014,242	614,434	18.3	756,159	580,393	7.6				1,046,259
Colorado	157,106	82,891	20.7	184,366	103,592	3.4	152,387	35,323	5.0	501,761
Connecticut	298,035	136,331	6.1	266,247	115,938	4.8				199,604
Delaware	81,076	38,116		224,923	111,986	12.3	102,873	37,618	3.9	246,037
Florida	1,042,471	519,285	11.6	292,018	153,304	6.1				398,953
Georgia	545,666	257,833	43.4	1,205,213	694,957	74.9	400,257	200,129	43.7	1,125,176
Idaho	293,347	172,516	26.2	225,220	157,021	7.7	38,149	23,587	2.8	285,992
Illinois	1,087,098	518,844	59.4	1,147,660	573,830	54.5	96,500	48,250	10.6	903,476
Indiana	594,207	291,074	39.8	1,128,455	531,071	46.7	189,600	94,800	6.4	941,303
Iowa	638,766	296,061	165.0	414,792	183,645	57.1	287,449	133,725	51.5	589,321
Kansas	806,405	404,547	106.4	1,615,909	813,454	90.5	460,565	230,282	36.9	1,039,609
Kentucky	1,157,864	321,329	83.2	1,249,010	417,668	37.2	74,213	24,400	5.6	324,331
Louisiana	538,142	265,678	20.6	7,700	3,850		289,362	138,761	21.5	666,649
Maine	77,540	38,770	3.6	235,218	117,609	10.6	16,850	2,714	.4	161,416
Maryland	579,966	289,808	20.7	226,410	113,205	1.8				345,564
Massachusetts	179,789	93,569	4.1	688,233	377,683	10.1				547,304
Michigan	1,217,047	599,924	76.3	786,598	393,299	24.8	315,970	157,985	11.2	639,267
Minnesota	1,599,861	797,479	217.1	990,917	501,197	86.9	300,776	149,988	30.1	590,308
Mississippi	829,715	414,858	46.5	1,474,046	670,888	59.0				425,220
Missouri	435,434	214,924	52.9	873,976	419,400	89.7	229,150	84,646	36.7	1,053,332
Montana	377,420	214,407	58.5	292,724	170,722	31.0	13,569	7,715	4.5	889,999
Nebraska	361,584	178,543	49.4	483,813	244,520	64.7	10,141	5,070	2.0	699,809
Nevada	324,082	265,067	28.6	96,404	64,163	4.6				231,959
New Hampshire	156,054	95,143	4.7	239,437	118,819	3.6				133,643
New Jersey	487,496	239,245	9.6	412,615	230,410	12.1	51,500	25,750	1.8	575,787
New Mexico	566,935	357,245	47.7	189,504	122,533	15.2				296,856
New York	959,667	469,927	28.8	1,135,012	608,144	14.9	245,600	122,800	.9	1,096,021
North Carolina	339,015	178,344	34.9	522,407	278,748	36.3	69,820	20,000	5.0	687,399
North Dakota	29,802	15,664	2.4	7,382	7,382		808,050	793,860	42.7	755,172
Ohio	1,785,007	889,427	57.6	901,610	498,775	14.5	76,000	38,000	2.0	1,373,997
Oklahoma	400,250	210,874	27.6	27,872	14,715	4.0	1,260,706	665,688	75.9	894,506
Oregon	459,181	244,030	41.8	482,318	255,794	28.5	30,482	18,000	1.3	366,285
Pennsylvania	2,015,130	996,906	37.6	513,989	256,587	10.4	73,588	36,794	1.8	766,798
Rhode Island	220,879	111,427	2.6	22,194	18,197					130,911
South Carolina	787,356	307,866	54.6	221,700	79,945	.5				397,720
South Dakota	36,665	22,152	15.2	4,156	4,156		1,143,430	1,047,600	114.5	762,018
Tennessee	468,207	232,400	14.1	1,337,954	668,977	46.0	158,042	79,021	4.5	770,269
Texas	1,439,323	699,860	146.2	584,379	285,797	48.9	43,500	21,700	4.8	2,249,798
Utah	186,949	123,241	17.0	153,666	101,709	6.8				337,043
Vermont	40,708	18,109	1.2	180,204	85,595	7.8	42,311	31,733	1.1	53,580
Virginia	406,206	191,396	16.2	346,766	155,305	4.5	25,150	12,575	3.9	676,811
Washington	293,361	170,221	17.8	440,348	201,425	13.7				413,564
West Virginia	427,292	215,822	19.8	301,364	150,069	6.4				501,897
Wisconsin	936,033	464,999	42.7	1,421,925	655,289	46.0	51,562	24,880	.8	596,353
Wyoming	365,617	158,064	18.8	508,423	218,112	34.3				220,633
District of Columbia	76,620	37,756	.9	2,558	1,279					163,600
Hawaii				2,375	2,375					334,875
Puerto Rico	170,027	77,541	7.5	73,642	38,810	3.1				217,968
TOTALS	29,551,971	14,660,801	1,935.4	25,918,830	13,113,100	1,195.2	7,394,028	4,473,973	546.6	30,089,846

PUBLICATIONS of the PUBLIC ROADS ADMINISTRATION

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Agency and as the Agency does not sell publications, please send no remittance to the Federal Works Agency.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.
Report of the Chief of the Bureau of Public Roads, 1932. 5 cents.
Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.
Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.
Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.
Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.
Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.
Report of the Chief of the Bureau of Public Roads, 1938. 10 cents.
Report of the Chief of the Bureau of Public Roads, 1939. 10 cents.
Work of the Public Roads Administration, 1940.
Work of the Public Roads Administration, 1941.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.
Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.
Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.
Part 4 . . . Official Inspection of Vehicles. 10 cents.
Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.
Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . . The Results of Physical Tests of Road-Building Rock. 25 cents.
No. 191MP. . Roadside Improvement. 10 cents.
No. 272MP. . Construction of Private Driveways. 10 cents.
No. 279MP. . Bibliography on Highway Lighting. 5 cents.
Highway Accidents. 10 cents.
The Taxation of Motor Vehicles in 1932. 35 cents.
Guides to Traffic Safety. 10 cents.
An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.
Highway Bond Calculations. 10 cents.
Transition Curves for Highways. 60 cents.
Highways of History. 25 cents.
Specifications for Construction of Roads and Bridges in National Forests and National Parks. 1 dollar.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.
No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.
No. 265T. . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Public Roads Administration upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP. . Bibliography on Highway Safety.
House Document No. 272 . . . Toll Roads and Free Roads.
Indexes to PUBLIC ROADS, volumes 6–8 and 10–21, inclusive.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

REPORTS IN COOPERATION WITH UNIVERSITY OF ILLINOIS

No. 303. . . Solutions for Certain Rectangular Slabs Continuous Over Flexible Support.
No. 304. . . A Distribution Procedure for the Analysis of Slabs Continuous Over Flexible Beams.
No. 313. . . Tests of Plaster-Model Slabs Subjected to Concentrated Loads.
No. 314. . . Tests of Reinforced Concrete Slabs Subjected to Concentrated Loads.
No. 315. . . Moments in Simple Span Bridge Slabs With Stiffened Edges.

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.
Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.
Act III.—Uniform Motor Vehicle Civil Liability Act.
Act IV.—Uniform Motor Vehicle Safety Responsibility Act.
Act V.—Uniform Act Regulating Traffic on Highways.
Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF MARCH 31, 1942

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$151,956	$151,436	2	4	2	$390,025	$386,803	5	2	2	$80,935	$80,935	3		5	$1,095,694
Arizona	184,378	184,361	1	1		125,274	116,583		1		13,255	13,255			2	234,779
Arkansas	471,125	469,820	5		7	171,046	169,336	1		2	12,359	12,359			4	683,335
California	836,454	646,882	2	1	5	870,516	868,399	8			9,914	9,914			3	2,319,499
Colorado	5,685	5,646				664,333	664,333	7			21,042	21,042			10	730,652
Connecticut	166,222	165,415	2			61,712	60,676		1		231,374	222,740		1		525,593
Delaware	89,725	89,725	1		1	191,599	189,867	1	1		508,406	321,785	2			179,564
Florida	120,961	120,961			20	843,067	840,887	8		6	82,492	82,432			14	870,795
Georgia	668,629	668,629	7	2	6	840,130	840,130	5	6	10	937,709	937,709	3	4	13	1,515,675
Idaho	27,850	23,603			3	322,273	313,602	4	1		4,189	4,189			2	427,671
Illinois	686,580	564,611	2		84	1,667,042	1,572,807	8	1	9	412,199	393,249	1	1	34	2,564,700
Indiana	607,483	607,483	6		26	471,234	465,525	2	1	17	79,950	79,950			23	1,170,601
Iowa	350,362	340,272	3	1	28	1,444,738	1,192,530	10	2	15	191,014	183,590			45	656,633
Kansas	69,711	69,206	2		7	756,728	756,728	9		4	135,254	131,170	3		6	1,314,353
Kentucky	1,109,873	1,107,509	9		1	441,711	441,711	4	1							411,382
Louisiana	6,965	6,965				586,220	586,220	8			481,835	480,667	4		4	924,798
Maine						367,591	367,591	2		1	7,739	7,739			3	285,059
Maryland	490,900	459,107	2	2	11	877,258	733,460	4		4	37,325	37,325			7	399,876
Massachusetts	346,270	335,829	1		1	791,642	790,770	5	2		763,830	763,830	2	1		1,250,554
Michigan	1,321,793	1,314,906	3	4	22	249,521	249,521	1		4	329,729	299,568	1		9	1,325,454
Minnesota	638,427	638,112	5	4	1	968,896	968,896	5	4	4	59,688	59,688		1	4	1,194,726
Mississippi	253,874	253,874	2	1	10	843,110	843,110	10			25,808	25,808			2	646,612
Missouri	120,702	120,702		2		1,970,283	1,514,863	6	2		416,991	416,991	2	2		1,489,049
Montana	141,549	141,549	2		2	99,778	99,778	1								733,051
Nebraska	179,203	178,161	2		22	1,142,413	1,142,413	21		1	13,020	13,020			3	473,945
Nevada	119,382	116,832	2		1	57,946	57,946	2			28,124	28,124			9	196,487
New Hampshire	265,368	265,069	4	1	3	96,565	96,240	2	1	2						313,961
New Jersey	852,812	850,349	4	1	2	622,904	497,354	3	1		354,985	295,560	1	2		941,191
New Mexico						68,342	68,342	1			259,103	252,068	3		1	522,006
New York	2,422,733	2,364,650	2	13		1,962,196	1,921,057	3	8		502,645	464,285		1		3,747,433
North Carolina	518,515	518,515	2	5	23	195,211	192,333		2	7	133,998	133,998	1		15	1,407,381
North Dakota	174,102	172,899	4			585,815	585,815	6								872,117
Ohio	1,545,117	1,529,900	8	1	7	2,780,072	2,456,350	10		2	403,460	199,330	1		6	1,638,941
Oklahoma	193,054	189,260	1		34	938,159	934,749	7		3	372,820	334,783	3		4	1,454,250
Oregon	419,536	355,255	4		3	203,552	187,715	2			10,433	10,433			3	447,396
Pennsylvania	1,907,614	1,876,905	14	1		3,016,940	3,002,411	14			359,074	359,074	2			2,526,669
Rhode Island	205,241	205,241		1		3,655	3,655									273,921
South Carolina	341,464	328,115	6		21	245,569	224,728	2	3	2	238,868	123,735	1	2	8	989,463
South Dakota	511,904	511,793	13		9	515,821	499,871	9			41,200	41,200	2			829,229
Tennessee	301,580	289,686	3	3		1,325,647	1,325,647	7	1		102,106	102,106		1	2	1,005,387
Texas	1,329,036	1,309,617	16			1,361,837	1,350,243	13		4	4,350	4,350			1	2,375,282
Utah	61,859	61,117	2		20	66,304	66,304	1		8	60,140	60,140			23	359,457
Vermont	18,683	18,671			4	322,869	293,090	2								103,978
Virginia	516,966	516,966	2	4	3	392,327	372,367	5	1	1	4,015	4,015			1	959,061
Washington	179,205	178,958	1	2	3	1,250,182	553,508	7			5,192	5,192			2	439,678
West Virginia	253,143	247,512	3		6	654,510	654,510	6	1	2	3,330	3,330			1	776,753
Wisconsin	468,542	451,282	2		39	568,623	567,635	3	2	6	15,484	15,484			3	1,674,169
Wyoming	481,271	466,615	6		1	1,974	1,974				8,417	8,416			6	429,928
District of Columbia	2,193	2,193				299,675	275,206		1							103,351
Hawaii	187,618	187,618	2			214,170	213,655	2								282,756
Puerto Rico	103,629	102,980	1			780,618	772,706	11								363,914
TOTALS	22,427,244	21,782,762	161	54	438	35,689,623	33,351,950	253	46	116	7,763,801	7,044,578	35	16	278	48,508,209

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D. M. BEACH, *Editor*

Volume 23, No. 3 May 1942

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE. By direction of the Commissioner of Public Roads, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

HILL-CLIMBING ABILITY OF MOTOR TRUCKS

BY THE DIVISION OF HIGHWAY TRANSPORT, PUBLIC ROADS ADMINISTRATION

Reported by CARL C. SAAL, Associate Highway Engineer-Economist

The problem of eliminating the traffic congestion that results from slow-moving vehicles on grades has been investigated in an exhaustive study of motor truck performance. The study was divided into three distinct but closely related parts: A study of the performance of new motor trucks in the best of condition; a study of the performance of used motor trucks in various stages of wear and having traveled various mileages; and a study of the hill-climbing ability and the driver behavior of a large number of motor trucks and passenger cars as they operate in every day traffic.

Actual grade tests, the most important part of the new truck study, were made on 30 new truck chassis. The results of these tests showed that for motor trucks even to approach reasonable speeds on grades: Grades must be reduced to 3 percent or less; or engine power must be more than doubled; or gross vehicle weights must be reduced excessively; or some combination of the three must be used that will still be costly to all interests involved and impossible of immediate application.

Deceleration tests were made on each new truck to determine the coefficients of tractive resistance. The most significant finding was that not only the total tractive resistance but also the unit resistance in pounds per 1,000 pounds varied appreciably with weight. The efficiency of the transmission of power was also determined for each new vehicle from the results of the actual grade tests. The evaluation of these two factors makes it possible to compute the performance of a motor truck from its specified characteristics with a fair degree of accuracy.

The results of the actual grade tests were also used to appraise cheaper and shorter methods for determining the grade ability of motor trucks. The acceleration method proved to be the most satisfactory and was adopted for use on the used truck study.

Tests on 17 used trucks of the same make and model as the new trucks tested showed that not over a 10 percent decrease in performance should be expected from wear and mileage. The results also proved definitely that trucks can be maintained so that their performance does not decrease with reasonable use.

The study of motor trucks in actual service under ordinary driving produced results that showed a 30 percent variation between the possible performance and the actual performance of vehicles of the same weight and capacity. Since not over 10 percent of this variation should be due to lack of maintenance, there remains a 20 percent variation that must be charged to improper operation of the vehicle. The shifting of gears at improper speeds was the principal reason for the variation in performance.

ELIMINATION of the traffic congestion that results from slow-moving vehicles on hills is a difficult problem. There are two possible solutions:

1. Facilities can be provided to enable other vehicles to pass the slow-moving vehicles.
2. The speed of the slow-moving vehicles can be increased.

Passing facilities can be provided by the construction of added lanes on hills and the building of highways with longer sight distances, but these solutions are localized in character. A more comprehensive solution and the one that is considered here is that of increasing speeds by reducing grades, increasing power, or reducing gross vehicle weight, or by a combination of these means. The purpose of the research reported herein is to determine if there is a reasonable minimum speed that will eliminate the congestion and, if so, how the three methods (grade reduction, weight reduction, and power increase) should be applied to make this speed a reality.

In order to supply the information that is needed to fulfill this purpose, in the spring of 1938 the Public Roads Administration in cooperation with the Automobile Manufacturers' Association, the Maryland Motor Truck Association, the National Bureau of Standards, and the Quartermaster Corps of the Army, inaugurated an exhaustive study of motor-truck performance. The study was divided into three distinct but closely related parts. A study of the performance of new motor trucks; a study of the performance of used motor trucks in various stages of wear and having traveled various mileages; and a study of the hill-climbing ability and the driver behavior of a large number of motor trucks and passenger cars as they operate in every day traffic. The results reveal some decidedly interesting and significant facts.

The primary purpose of the new truck study was to determine the maximum grade performance that can be expected from various motor trucks. It was desired to obtain accurate information on which to consider the feasibility of imposing performance requirements and the facts needed to determine what methods can best be used to enable motor trucks to maintain reasonable speeds on grades.

It was also proposed to determine, if possible, a means for computing with some degree of accuracy the performance of a motor truck from its specified characteristics. In order to do this it was necessary to evaluate two important factors—the coefficient of tractive resistance and the mechanical efficiency of the transmission of power.

Another purpose of this study was to develop a method that could be used in the used-truck study to determine the performance accurately in less time and with less expense. After experimenting with several methods, one that makes use of values of acceleration was determined to be the most suitable.

The field tests were conducted in the vicinity of Baltimore, Md., with the Holabird Quartermaster Corps Depot as the base of operations. The facilities of the depot were made available for the tests, including personnel, equipment, a gasoline supply, use of the machine shop and dynamometer laboratory, and storage space. The Johns Hopkins University furnished office space for the analysis work, which kept pace with the field work.

LIGHT, MEDIUM AND HEAVY TRUCKS TESTED

The new trucks and tractor trucks involved in the tests were supplied by the manufacturers. The chassis tested were divided into three size groups—light, medium, and heavy. The light group included vehicles rated as 1½ tons; the medium group included vehicles 2 tons to less than 5 tons; and the heavy group included vehicles 5 tons and over. The models selected for each capacity group were those widely used currently and as nearly alike in piston displacement as possible, so that they would represent the vehicles now in common use and also be comparable as to power output. The light, medium, and heavy chassis were generally equipped with engines of approximately 230-, 300-, and 400-cubic inch piston displacement, respectively. The vehicles tested are described in table 1.

The manufacturers furnished expert drivers and mechanics to operate the vehicles and keep them in the best of condition throughout the tests. Every possible care was exercised to insure that the maximum performance was obtained. In addition, each manufacturer was represented during the tests by an engineer who inspected the operations and results to make certain that the maximum performance was being measured. The motors of the new chassis were thoroughly run in before the start of the tests.

A semitrailer equipped with a platform body with 1½-foot side boards and a platform truck body with 2-foot side boards were used for all the tests on the tractor trucks and single-unit trucks, respectively. Although the frontal area of the bodies used was not as great as that of the van type bodies in common use on the highway, any variance in performance that might result from the use of these bodies would not be significant for the speeds involved, especially since the tests were not made if there was a strong head or tail wind.

All of the trucks and tractor trucks were tested with gasoline from pumps operated by the Holabird Quartermaster Corps Depot. The National Bureau of Standards made tests on a sample of gasoline from each tank car that was emptied into the storage tanks from which the gasoline was pumped, in order to determine if the quality of the gasoline remained constant for the duration of the tests. A quart sample was obtained each time a vehicle was serviced with gasoline. A composite sample for a given tank car was then formed by blending the several quart samples at a temperature below 0° centigrade.

TABLE 1.—*Description of trucks and tractor-trucks tested in new truck study*

Make	Model	Year	Engine				Gear ratios						Tire size
			Piston displacement	Maximum torque	Maximum horsepower at r. p. m. given	Rear axle	Transmission						
							1	2	3	4	5		
			Cu. in.	*Lb.-ft.*									
LIGHT TRUCKS[1]													
G. M. C.	T16B	1938	230	172	86-3500	[2] 5.14 7.15	7.23	3.48	1.71	1.00		7.50×20
International	D30	1938	232	170	81-3200	6.17	6.40	3.09	1.69	1.00		7.50×20
White	700	1938	250	175	76-2800	5.83	6.40	3.09	1.69	1.00		7.50×20
Dodge	TF37	1939	228	158	80-3200	6.33	6.40	3.09	1.69	1.00		7.00×20
Chevrolet	VD	1939	216	170	78-3200	6.17	7.23	3.48	1.71	1.00		32×6
Diamond T	404	1939	245	170	77-3200	6.33	6.34	3.28	1.65	1.00		32×6
LIGHT TRACTOR TRUCKS[1]													
G. M. C.	T16A	1938	230	172	86-3500	[2] 5.14 7.15	7.23	3.48	1.71	1.00		7-50×20
International	D30	1938	232	170	81-3200	6.66	6.40	3.09	1.69	1.00		7.50×20
White	700	1938	250	175	76-2800	6.80	6.15	3.58	1.86	1.00	0.77	7.50×20
Dodge	TF36	1939	228	158	80-3200	6.33	6.40	3.09	1.69	1.00		7.00×20
Chevrolet	VB	1939	216	170	78-3200	6.17	7.23	3.48	1.71	1.00		32×6
Federal	15	1939	228	155	72-3000	6.67	6.40	3.09	1.69	1.00		7.50×20
Ford	98T	1940	239	170	95-3600	6.67	6.40	3.09	1.69	1.00		8.75×18
MEDIUM TRUCKS[1]													
G. M. C.	T33H	1938	286	220	99-2800	6.43	7.58	4.38	2.40	1.48	1.00	10.50×20
Mack	EH	1938	310	210	90-3000	8.59	6.10	3.48	2.04	1.00	.768	9.00×20
International	D50	1938	298	218	94-2800	7.16	6.52	3.72	1.92	1.00	.823	9.00×20
White	710	1938	318	245	110-3000	7.14	6.06	3.50	1.80	1.00	.799	9.00×20
MEDIUM TRACTOR TRUCKS[1]													
G. M. C.	T33H	1938	286	220	99-2800	6.43	7.58	4.38	2.40	1.48	1.00	10.50×20
Mack	EH	1938	310	210	90-3000	8.59	6.10	3.48	2.04	1.00	.768	9.00×20
International	D50	1938	298	218	94-2800	7.16	6.52	3.72	1.92	1.00	.823	9.00×20
White	710	1938	318	245	110-3000	7.14	6.06	3.50	1.80	1.00	.799	9.00×20
Diamond T	614	1939	320	223	81-2500	[2] 6.43 8.74	7.58	4.38	2.40	1.48	1.00	9.00×20
Federal	29	1939	320	244	95-2800	7.40	7.58	4.38	2.40	1.48	1.00	9.75×20
HEAVY TRACTOR TRUCKS[1]													
G. M. C.	T46B	1938	400	304	120-2500	[2] 5.62 7.65	6.63	3.20	1.70	1.00	.74	10.50×20
Mack	BM	1938	415	271	108-2400	8.64	6.90	3.91	1.89	1.00	.794	10.50×20
International	DR70	1938	401	308	114-2600	9.03	6.98	3.57	1.89	1.00	.825	11.25×20
White	750	1938	362	280	116-3000	7.14	7.00	3.97	1.80	1.00	.788	10.50×20
MEDIUM AND HEAVY TRACTOR TRUCKS[3]													
G. M. C.	602	1939	213	263	72-2000	[2] 6.43 8.74	6.21	3.52	1.81	1.00	.77	9.75×20
Dodge	TKD	1939	331	226	95-2600	[2] 6.14 8.35	7.58	4.38	2.39	1.48	1.00	9.75×20
Mack	ED	1940	519	383	131-2000	[4] 7.54	6.74 9.30	3.82 5.27	1.92 2.65	1.00 1.38	.78 1.08	10.50×20

[1] Gasoline engines.
[2] 2-speed axles.
[3] Diesel engines.
[4] Auxiliary transmission.

Tests were made to determine the A. S. T. M. octane number, the Reid vapor pressure, the sulfur content, and the distillation range. The results of these tests are contained in table 2. The octane number is 71 for 4 of the samples and 72 for the other 16 samples. The results definitely show that the gasoline used during the period of study was of uniform quality.

Each of the new motor trucks was subjected to an exhaustive series of tests. The grade ability was determined by actual road tests made by placing various loads on the vehicles and observing the sustained speeds maintained on known gradients, and by acceleration tests that measured the drawbar force available at various road speeds over the entire useful speed range of each gear. The tractive resistance, an important variable in any consideration of grade ability, was determined for each vehicle by deceleration tests that measured the force opposing the motion of the vehicle when coasting in neutral on a level grade. Dynamometer tests were made to determine the power output of each engine so that the certified power and torque curves submitted by the manufacturer could be verified.

The actual grade tests, the most important part of the new truck study, involved the testing of the 10 single-unit trucks and the 20 tractor-truck semitrailers on several uniform grades. At the start of the study each truck was tested on grades of 3.2, 4.0, 4.5, 6.0, and 7.0 percent. However, after testing several units sufficient data were obtained to prove that the performance for one grade could be accurately converted to that for another grade. Thereafter the tests were conducted on uniform grades of 4.5 and 6.0 percent.

SUSTAINED SPEEDS DETERMINED FOR VARIOUS LOADS IN EACH GEAR

Known loads were placed on each truck, and the maximum sustained speed that it could maintain on a known grade was determined. The maximum gross vehicle weight that the truck could pull up the grade in a given gear at a constant speed was determined by trial, using the performance indicated by an ability formula as a guide. Starting with the maximum weight that could be hauled in a given gear, the load was decreased by 1,000- or 500-pound decrements and the maximum sustained speed measured for each gross vehicle weight. The weight was decreased until the sustained speed increased to a value that corresponded to an engine speed that approximated the maximum recommended by the manufacturer or, if the motor was governed, to the governed engine speed. Vehicles equipped with governors were tested both with and without the governors in operation. When the tests were completed in one gear, they were continued in the gear with the next lower gear ratio. The tests were continued until the empty weight of the truck or combination was reached.

Several test runs were required to determine the sustained speed for each gross weight because the grades used were not long enough to slow down a vehicle to a crawl speed on the first trial. All of the test runs were made with full throttle. The truck was driven onto the grade at a speed estimated to be that which could be sustained over the entire length of grade. An observer in the cab of the truck recorded the speed indicated by the truck speedometer at the start and the end of the test run, and determined whether a sustained speed was maintained. If the vehicle accelerated or decelerated on the first test run, the grade was entered on the next run at about the speed indicated when the truck left the grade on the previous run. When this procedure finally resulted in a speed that appeared to be the one that the truck could maintain over the entire length of the grade, a check run was made to verify it.

Typical field data for a 6-percent grade are shown in table 3. For third gear, after trying weights of 19,000 and 18,000 pounds, the maximum gross vehicle weight that could be carried at a sustained speed was determined to be 17,500 pounds. From this point the load was decreased by decrements of 1,000 pounds to a gross weight of 11,500 pounds, at which point a road speed was observed that corresponded to the maximum engine speed recommended. For the gross weight of 13,500 pounds, five test runs (46 to 50 inclusive) were required to determine the speed that could be sustained. The grade was entered on the first run at 30 miles per hour, and the truck decelerated to a speed of about 27 miles per hour. On the second run, the truck entered the grade at 25 miles per hour and accelerated to a speed of 27 miles per hour. These two runs indicated that the sustained speed for this particular weight was about 27 miles per hour. The following three runs estab-

TABLE 2.—*Results of tests made on gasoline used in making motor-truck performance studies*

Type of test	Sample No.																			
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
A. S. T. M. octane No.	72	72	71	71	71	72	72	72	72	72	71	72	72	72	72	72	72	72	72	72
Reid vapor pressure *lb/sq. in*	8.6	8.3	8.0	8.0	7.7	7.7	8.2	8.1	8.9	8.9	8.4	7.7	8.2	8.6	9.0	9.9	9.5	9.2	9.1	8.5
Sulfur, *percent*	.09	.09	.09	.09	.09	.09	.07	.06	.06	.07	.09	.10	.10	.10	.12	.11	.10	.10	.10	.10
Distillation:																				
First drop °C	37	37	35	37	37	37	36	35	35	35	38	37	37	35	39	33	33	32	34	34
5 percent distilled °C	53	53	53	54	55	55	53	54	50	50	53	52	53	51	51	49	50	48	49	50
10 percent distilled °C	62	62	61	64	64	64	62	63	59	58	60	59	60	58	58	57	58	56	58	57
15 percent distilled °C	70	70	70	72	72	73	71	71	68	67	67	65	67	66	66	66	67	65	66	65
20 percent distilled °C	78	78	79	80	80	81	80	80	77	76	74	71	74	73	73	75	75	74	74	73
30 percent distilled °C	94	94	94	95	95	96	96	96	94	92	88	84	89	87	87	92	92	91	91	90
40 percent distilled °C	107	108	108	109	108	109	110	110	109	107	102	97	102	101	100	106	106	106	107	106
50 percent distilled °C	120	120	120	121	121	121	123	123	122	122	115	109	115	114	112	118	118	119	120	121
60 percent distilled °C	133	130	133	133	133	133	133	135	134	134	126	121	124	124	124	130	130	130	130	133
70 percent distilled °C	145	147	146	146	147	145	146	146	146	146	139	134	138	138	135	142	137	140	144	146
80 percent distilled °C	159	162	162	163	162	162	160	160	161	159	155	150	155	155	152	158	159	158	159	160
90 percent distilled °C	181	182	182	181	179	182	178	178	178	177	173	170	176	176	173	177	178	179	178	179
95 percent distilled °C	198	200	197	198	195	197	193	192	193	192	192		193	193	191	196	196	195	194	194
End Point °C	215	212	214	214	204	214	208	204	202	203	203	203	205	203	202	204	205	203	204	203
Recovery *percent*	96.9	96.9	97.5	97.4	96.9	97.5	97.5	97.5	97.1	97.5	97.1	97.8	97.3	97.5	97.5	96.8	96.7	96.9	97.5	97.7
Residue do	1.1	1.3	1.2	1.1	1.7	1.3	.9	1.1	1.0	1.1	1.0	1.1	1.1	1.1	1.2	.9	1.0	1.0	.9	1.0
Loss do	2.0	1.8	1.3	1.5	1.4	1.2	1.6	1.4	1.9	1.4	1.6	1.1	1.6	1.4	1.3	2.3	2.3	2.1	1.6	1.3
Barometric pressure *mm. of mercury*	752	752	760	754	753	748	747	747	746	746	746	744	760	760	760	749	757	757	757	757

lished the sustained speed at approximately 27 miles per hour. The same procedure was used for each weight.

The radius of the driving wheels when loaded was measured in the field for each test weight as shown in table 3. This measurement must be determined in order to relate the power output at the engine to that available at the driving wheels. It was measured by means of an adjustable arm attached to a meter stick. A small level was mounted on the arm to insure accuracy of measurement.

TABLE 3.—*Sample of field data recorded for a tractor-truck semitrailer on a 6-percent grade*

Test No.	Gear used	Gross vehicle weight	Speedometer reading			Radius of driving wheels when loaded			Remarks
			Enter	Leave	Sustained	Right	Left	Average	
		Pounds	*M. p. h.*	*M. p. h.*	*M. p. h.*	*Inches*	*Inches*	*Inches*	
24	2	22, 750	14	16+	16+	17. 13	17. 13	17. 13	
25	2	22, 750	16+	16+	16+				
26	2	22, 750	16+	16+	16+				
27	3	19, 000	12	4	-------	17. 25	17. 15	17. 20	Had to shift gears.
28	3	18, 000	12	7	--------	17. 20	17. 20	17. 20	Do.
29	3	18, 000	9	4	--------				
30	3	17, 500	12	10−	10−	17. 20	17. 24	17. 22	
31	3	17, 500	9	9+	9+				
32	3	17, 500	8	9	9				
33	3	16, 500	15	14	--------	17. 25	17. 31	17. 28	
34	3	16, 500	12	13+	13+				
35	3	16, 500	14	13+	13+				
36	3	16, 500	13+	13+	13+				
37	3	15, 500	20	20	20	17. 41	17. 31	17. 36	
38	3	15, 500	22	20	--------				
39	3	15, 500	18	19+	--------				
40	3	15, 500	19	20−	20−				
41	3	14, 500	25	24	24	17. 41	17. 35	17. 38	
42	3	14, 500	22	23+	--------				
43	3	14, 500	24	24	24				
44	3	14, 500	24	24	24				
45	3	14, 500	27−	25−	--------				
46	3	13, 500	30	27+	--------	17. 52	17. 44	17. 48	
47	3	13, 500	25	27	--------				
48	3	13, 500	27+	27	27				
49	3	13, 500	27−	27−	27−				
50	3	13, 500	27−	27−	27−				
51	3	12, 500	28	29+	----	17. 52	17. 48	17. 50	
52	3	12, 500	30	29+	--------				
53	3	12, 500	29+	29+	29+				
54	3	12, 500	29+	29+	29+				
55	3	12, 500	30+	29+	--------				
56	3	11, 500	28	31	--------	17. 52	17. 52	17. 52	
57	3	11, 500	31	31+	31+				
58	3	11, 500	32−	32−	32−				

The speed indicated by the truck speedometer was used only in the field to determine that a sustained speed had been maintained on the grade. The speed was measured more accurately by a time-distance recorder which produced a record of time and distance that was used to determine approximately instantaneous speeds for each test run. The recording unit consisted of a chronograph with three magnetic recording styles. Two brushes, riding on a two-point cam mounted on the axle of a bicycle wheel that was attached to the truck bumper (see cover illustration), were wired in series with one of the styles and caused each half revolution of the wheel to be recorded. A clock with six contacts on the second hand shaft, wired in series with another of the styles, created a time record at 10-second intervals. A telegraph key, wired in series with a third style, was used to mark the beginning and the end of the test course. The recording tape was driven by a spring driven motor with a governor, and the tape speed was kept constant for the duration of each test run. Figure 1 shows the wiring diagram for the time-distance recorder.

Figure 2 shows a section of an actual tape record. The approximate instantaneous speed at any point was calculated by dividing the distance represented by a given number of revolutions by the time required to travel that distance. For example, at the 5-second point marked on the tape record (fig. 2), two complete revolutions of the wheel were used in the computations. Since the circumference of the wheel is 6.5 feet, the distance traveled in two revolutions is 13 feet. The tape speed is 0.85 inch per second, which makes each inch on the tape equivalent to 1.177 seconds. Since the two revolutions occupied 0.70 inch on the tape, it took the vehicle 0.824 second to travel the 13 feet. The speed is then 15.8 feet per second or 10.8 miles per hour.

The circumference of the bicycle wheel used with the recorder was determined by measuring the distance covered in 10 complete revolutions of the wheel and dividing by 10. The measurements were made at the beginning of each series of tests and also several times during the tests to insure that there was no change in the circumference. The distance record obtained with the fifth wheel and recorder was never in error more than 1 foot in 1,000 feet. This accuracy was obtained by underinflating the tire so that the wheel did not bounce off the road surface.

Approximately instantaneous speeds were computed at short time intervals for each test run. From the record of instantaneous speeds it was possible to ascertain definitely whether the truck was accelerating, decelerating, or traveling at a uniform speed. After the runs with sustained speeds had been selected by an inspection of the instantaneous speeds, the uniform speed was computed by dividing the distance traveled by the total time required for the run.

The record of instantaneous speeds for the five runs (Nos. 46 to 50, inclusive, in table 3) that were used to demonstrate the method used in the field to obtain the sustained speed is shown in table 4. Inspection of the instantaneous speeds reveals that a constant speed was maintained on runs 49 and 50. Runs 46, 47, and 48 show that the vehicle was accelerating or decelerating for the greater part of the distance. Actually the sustained speed was 25.7 miles per hour instead of the 27 miles per hour indicated by the truck's speedometer.

TABLE 4.—*Record of instantaneous speeds for 5 test runs on a 6-percent grade*

Seconds	Run 46	Run 47	Run 48	Run 49	Run 50
	M. p. h.	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*
0	29.0	24.7	26.5	25.9	25.7
2	28.7	24.7	26.5	25.9	25.7
4	28.0	24.7	26.5	25.9	25.7
6	28.0	25.0	26.8	25.9	25.7
8	28.0	25.0	26.5	25.9	25.7
10	27.7	25.0	26.5	25.9	25.7
12	27.7	25.0	26.5	25.7	25.7
14	27.4	25.3	26.5	25.7	25.7
16	27.4	25.3	26.5	25.7	25.7
18	27.1	25.3	26.5	25.7	25.7
20	27.1	25.3	26.2	25.7	25.7
22	27.1	25.7	26.2	25.7	25.7
24	26.8	25.7	26.2	25.7	25.7
26	26.8	25.7	26.2	25.7	25.7
28	26.5	25.3	26.2	25.7	25.7
30	26.5	25.3	25.9	25.7	25.7
32	26.5	25.7	25.9	25.7	25.7
34	26.8	25.7	25.6	25.7	25.7
36	[1] 26.5	25.7	[2] 25.9	25.7	25.7
38	--------	[3] 25.7	--------	[4] 25.7	[5] 25.7

[1] At 36.03 seconds.
[2] At 37.58 seconds.
[3] At 38.9 seconds.
[4] At 38.35 seconds.
[5] At 38.55 seconds.

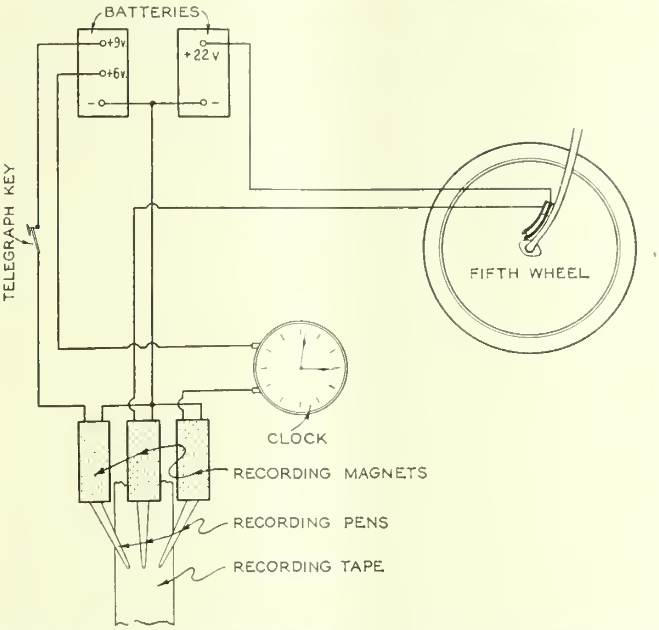

FIGURE 1.—WIRING DIAGRAM FOR TIME-DISTANCE RECORDER.

GRADE PERFORMANCE CORRECTED TO STANDARD ATMOSPHERIC CONDITIONS

In order to place the results of the tests on the 30 single unit trucks and combination units on a comparable basis, the grade performance in every case was corrected to standard atmospheric conditions. The formula used to compute the correction factor is as follows:

$$CF=\frac{29.92}{P_0-H}\sqrt{\frac{T_0}{520}} \qquad (1)$$

where

CF=correction factor,
P_0=observed barometric pressures in inches of mercury,
H=water vapor pressure in inches of mercury, and
T_0=observed absolute air temperature in degrees Fahrenheit.

The information required to compute this factor was obtained in the field, with the exception of the barometric pressure which was obtained from the Weather Bureau in nearby Baltimore. In addition to the dry and wet bulb temperatures that were used to obtain the water vapor pressure, the wind direction and velocity were also recorded. Tests were not made if the head wind exceeded 10 miles per hour or the tail wind 15 miles per hour. A miniature weather station consisting of a thermometer, a hand asperated psychrometer and an anemometer and wind vane, was operated in the center of each test section. The observations were taken during each test run.

The correction factor was computed for each test run on which a sustained speed was observed and applied to the gross vehicle weight, since for any given speed the power developed by the test vehicle is directly proportional to the weight carried. The corrected performance of a tractor-truck semitrailer operating in third gear on a 6-percent grade is shown in table 5. In this particular case the correction factors are near unity, indicating that the tests were made at approximately standard atmospheric conditions. The last two columns of table 5 contain the corrected gross vehicle weights and the speeds that the vehicle can sustain with those weights.

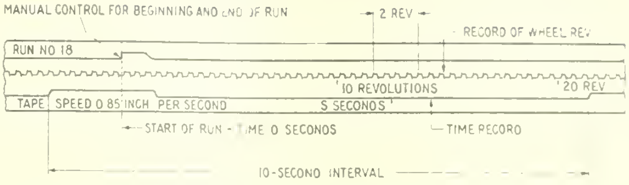
MANUAL CONTROL FOR BEGINNING AND END OF RUN
2 REV
RECORD OF WHEEL REV
RUN NO 18
10 REVOLUTIONS
20 REV
TAPE
SPEED 0 85 INCH PER SECOND
5 SECONDS
START OF RUN - TIME 0 SECONDS
TIME RECORD
10-SECOND INTERVAL

FIGURE 2.—A SECTION OF CHRONOGRAPH TAPE RECORD.

The performance of the tractor-truck semitrailer, used as an example above, is shown graphically in figure 3. The performance curves for each transmission gear were obtained by plotting the corrected gross vehicle weight in thousands of pounds as the abscissae, and the speed in miles per hour as the ordinates. The curves show the performance of the vehicle on a 6-percent grade. Similar curves were plotted for each vehicle for grades ranging from 3 to 7 percent, inclusive.

TABLE 5.—*Actual grade performance corrected to standard temperature and barometric pressure*

[Make—A. Date—10-31-38. Model—X (without governor). Gear—third. Grade—6 percent]

Run No.	Time of day, p. m.	Relative humidity	Temperature: Dry bulb	Temperature: Dry-wet	Temperature: Dew point	Vapor pressure (H)	Barometric pressure (P_0)	P_0-H	Correction factor	Actual GVW	Corrected GVW	Sustained speed
		Pct.	° F.	° F.	° F.	In. mercury	In. mercury			Lb.	Lb.	M. p. h.
31	2:25	41	62	12.0	38	0.23	30.07	29.84	1.003	17,500	17,600	9.2
32	2:35	41	61	12.0	36	.21	30.07	29.86	1.001	17,500	17,500	8.7
35	2:55	42	62	12.0	38	.23	30.07	29.84	1.003	16,500	16,500	13.1
36	3:00	42	62	12.0	38	.23	30.07	29.84	1.003	16,500	16,500	13.1
37	3:05	42	61	11.5	38	.23	30.07	29.84	1.002	15,500	15,500	19.6
40	3:10	42	60	11.5	36	.21	30.07	29.86	1.001	15,500	15,500	18.9
43	3:20	42	61	11.5	38	.23	30.07	29.84	1.002	14,500	14,500	23.2
44	3:25	42	61	11.5	38	.23	30.07	29.84	1.002	14,500	14,500	23.2
49	3:35	43	59	11.0	36	.21	30.07	29.86	1.000	13,500	13,500	25.8
50	3:40	43	58	10.5	36	.21	30.07	29.86	.999	13,500	13,500	25.6
52	3:55	43	56	10.5	33	.19	30.07	29.88	.996	12,500	12,500	28.5
53	4:00	43	56	10.5	33	.19	30.07	29.88	.996	12,500	12,500	28.3
54	4:05	43	55	10.0	33	.19	30.07	29.88	.995	12,500	12,400	28.2
55	4:10	43	56	10.5	33	.19	30.07	29.88	.996	12,500	12,500	28.8
57	4:20	43	55	10.0	33	.19	30.07	29.88	.995	11,500	11,400	30.5
58	4:25	43	54	10.0	32	.18	30.07	29.89	.994	11,500	11,400	30.6

Since the trucks were tested on only 4½- and 6-percent grades, it was necessary to convert the ability observed for the 4½-percent grade to that for 3-, 4-, and 5-percent grades, and the ability for the 6-percent grade to that for 5- and 7-percent grades. The ability on a 5-percent grade was derived from the two sources in order to furnish a check on the accuracy of the methods used in the conversion.

The conversion is based on the assumption that a given vehicle will produce a tractive effort on one grade equal to that on another grade when the vehicle is operating in identical gears at like speeds. The tractive effort produced by a vehicle traveling over the grade at a uniform speed is equal to the component of the weight along the grade line plus the tractive resistance. Thus,

$$TE=GVW(f+g) \qquad (2)$$

where

TE=tractive effort in pounds,
GVW=gross vehicle weight in pounds,
f=coefficient of tractive resistance in pounds per pound, and
g=grade in feet of rise per foot.

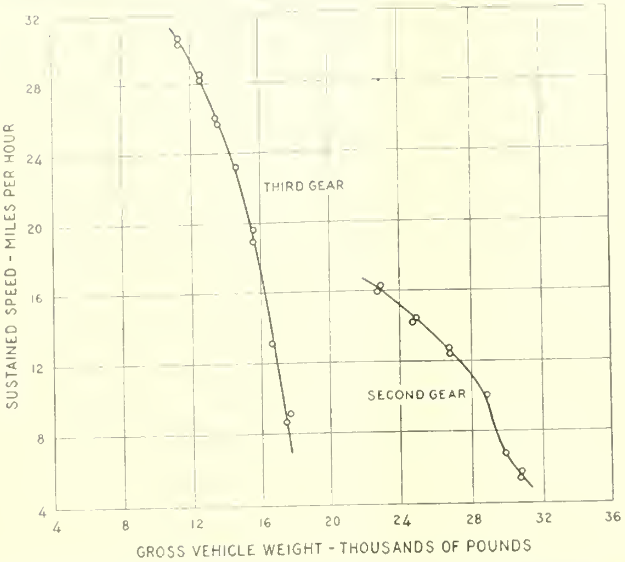

Figure 3.—Performance of Typical Tractor-Truck Semitrailer on a 6-Percent Grade, Without Governor, at Standard Atmospheric Conditions.

When the tractive effort on one grade is equaled to that for another grade, the following results:

$$GVW_1(f_1+g_1)=GVW_2(f_2+g_2)$$

or

$$GVW_2=\frac{GVW_1(f_1+g_1)}{(f_2+g_2)}\text{-------- ---}(3)$$

From figure 3, 13,500 pounds were hauled by the tractor-truck semitrailer on a 6-percent grade at 26 miles per hour. Assume that it is desired to determine what weight this truck can haul on a 5-percent grade at the same speed. The coefficient of tractive resistance determined by deceleration tests, described later in this report, is 0.0135 pounds per pound of weight for the speed and weight in question. Therefore, the tractive effort developed on the 6-percent grade at 26 miles per hour is 13,500 (0.06+0.0135) or 992 pounds and it is available at the same speed on any grade. This value represents the term $GVW_1(f_1+g_1)$ in the above equation. Since tractive resistance varies with load, there are two unknowns (GVW_2 and f_2) in the equation, therefore the value for f_2 cannot be directly determined until the approximate weight is known. This means that trial computations must be made to determine GVW_2. The first computation is made assuming f_2 to be 0.0135 pounds per pound, the coefficient for GVW_1. Using the GVW_2 determined on the trial computation another coefficient is obtained and substituted in the formula to obtain a second GVW_2. When a value for GVW_2 is computed that has a coefficient approximately the same as the one used to obtain it, the conversion has been completed.

For example, the first computation for a 5-percent grade would be:

$$GVW_2=\frac{992}{(0.05+0.0135)}=15{,}600\text{ pounds.}$$

The coefficient of tractive resistance for 15,600 pounds and 26 miles per hour is 0.0129 instead of 0.0135 pounds per pound as assumed. A second computation then follows:

$$GVW_2=\frac{992}{(0.05+0.0129)}=15{,}800\text{ pounds.}$$

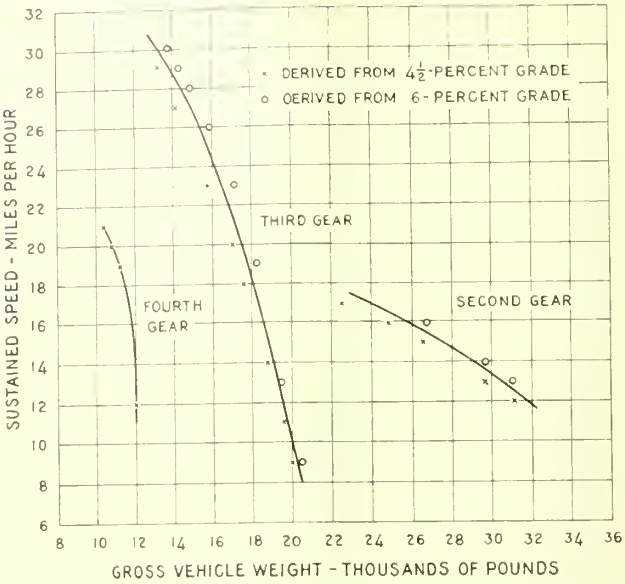

Figure 4.—Performance of Typical Tractor-Truck Semitrailer on a 5-Percent Grade, Without Governor, at Standard Atmospheric Conditions.

As the coefficient of tractive resistance for a load of 15,800 pounds is 0.0129, the same as that for 15,600 pounds, the conversion has been completed. The truck, then is capable of a speed of 26 miles per hour with a weight of 15,800 pounds on a 5-percent grade. Figure 4 shows the grade ability of this truck on a 5-percent grade. The curve is determined from points obtained by converting its performance on 4.5- and 6-percent grades to that for a 5-percent grade. The points derived from the two sources are in close agreement.

PERFORMANCE OF TRUCKS AND TRACTOR-TRUCKS SAME FOR EQUAL WEIGHTS

While the actual grade ability of an individual truck such as that shown in figures 3 and 4 is needed in order to evaluate the mechanical efficiency of the transmission of power and to determine the worth of other methods for measuring performance, both theoretical and experimental, it is the average performance of the vehicles in each capacity group that is more applicable to the existing problems.

The average performance for vehicles in the light, medium, and heavy capacity groups is shown in figures 5, 6, and 7, respectively. The results for the single-unit trucks and the tractor-truck semitrailers are combined, since there is no appreciable difference in performance for identical weights. These charts show the speeds that can be maintained with various gross vehicle weights on grades up to 7 percent.

The results shown in figures 5, 6, and 7 are for motor trucks operated without governors. Since a governor might affect the performance of a vehicle, the vehicles that were equipped with governors were tested both with and without the governors to determine if there was any appreciable difference in the performance of the vehicles when operated under the two conditions. The tests with the governor were limited to one grade only.

For a given vehicle the effect of the governor on the power output was negligible until the engine speed approached to within several hundred revolutions per minute of the governed engine speed. However, there was no consistency between the governors as to the range of engine speed for which there was a decrease in

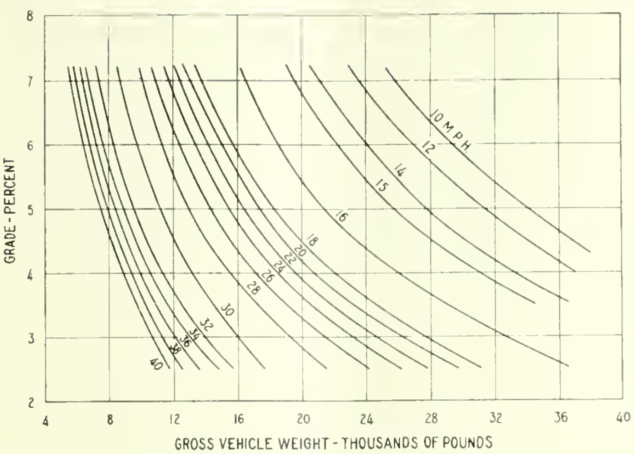

FIGURE 5.—GRADE ABILITY OF LIGHT TRUCKS AND TRACTOR-TRUCK SEMITRAILERS, WITHOUT GOVERNOR.

power output, even for the same make of governor. The average performance of the heavy tractor-trucks operated with governors is shown in figure 8. A comparison between the results shown in figures 7 and 8 indicates that the maximum average performance was obtained without the governor. For example, for common weights of 20,000, 30,000, and 40,000 pounds and a common grade of 4 percent, the sustained speed is 31, 21, and 17 miles per hour, respectively, without the governor, and 29, 19, and 15 miles per hour, respectively, with the governor. The variation measured in speed is not great. However, for a given speed the variation in terms of weight may be as much as 5,000 pounds.

When using the results of the actual grade tests it is important to remember that the performance is an average value determined from the maximum performances obtained for individual vehicles. In order to use the information intelligently it is necessary to know how much the individual performances deviate from the average performance. Table 6 lists the percentages by which the sustained speeds of the individual vehicles deviate from the average sustained speed for several gross vehicle weights. The average deviation varies in most cases between 5 and 10 percent. The dispersion indicated is within the accuracy desired for the uses that will be made of the average performance.

TABLE 6.—*Deviation of the performance of individual motor trucks from the average performance for each capacity group*

Type of vehicle	Average deviation, in percentage of the speed, for vehicle weights of—				
	12,000 pounds	18,000 pounds	24,000 pounds	30,000 pounds	36,000 pounds
Light trucks and tractor trucks	5.8	7.9	6.6	8.1	
Medium trucks and tractor trucks	2.3	8.1	8.0	9.0	4.9
Heavy tractor trucks		7.2	9.2	7.1	14.2

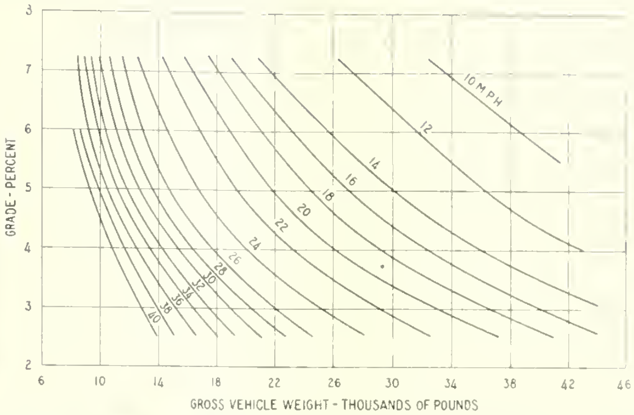

FIGURE 6.—GRADE ABILITY OF MEDIUM TRUCKS AND TRACTOR-TRUCK SEMITRAILERS, WITHOUT GOVERNOR.

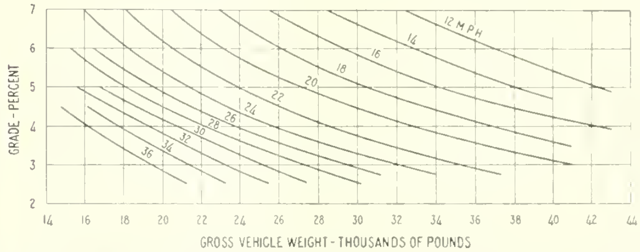

FIGURE 7.—GRADE ABILITY OF HEAVY TRACTOR-TRUCK SEMITRAILERS, WITHOUT GOVERNOR.

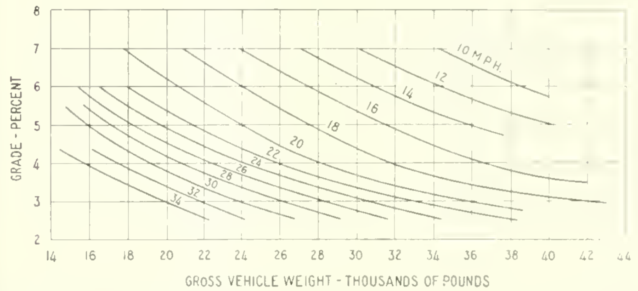

FIGURE 8.—GRADE ABILITY OF HEAVY TRACTOR-TRUCK SEMITRAILER, WITH GOVERNOR.

The most significant fact revealed by the performance charts is that the increase in speed that results from a reduction of grade is small. The average gross vehicle weight rating for light tractor-trucks is about 24,000 pounds. With this gross weight the average light tractor-truck is capable of 23, 17, 15, 14, and 12 miles per hour on 3-, 4-, 5-, 6-, and 7-percent grades, respectively. Reduction from a 7- to a 4-percent grade would raise the speed only 5 miles per hour, whereas reduction from a 4- to a 3-percent grade would raise the speed 6 miles per hour. It is evident that grades must be reduced to 3 percent or less if there is to be a marked betterment in performance.

Another very significant fact is that there must be a large reduction in gross weight to increase the speed of a vehicle appreciably. To raise the speed of the same light tractor-truck from 14 to 17 miles per hour on a 6-percent grade (the increase that would be obtained by reducing the grade from 6 to 4 percent) the gross weight must be reduced about 7,000 pounds. Since all of this reduction would come from the payload which originally would be about 12,000 pounds, there is a net reduction in load of almost 60 percent.

An increase in the power of the motor has also been mentioned as a means of bettering the speed. In order to increase the speed of the vehicle with a 24,000-pound gross weight from 14 to 17 miles per hour (an increase that would require a payload reduction of 60 percent or a 2-percent grade reduction) a motor almost as powerful as that used in a heavy truck would be required. This means about a 45-percent power increase to obtain an increase of 3 miles per hour in road speed on a 6-percent grade. The power would have to be at least doubled to provide what could be considered a reasonable speed.

It appears that it will be impracticable to apply any one method to obtain a reasonable speed. However, there is the possibility that a combination of the three methods can be used. For example, if a 25-percent reduction in payload is made, the weight of the light truck originally weighing 24,000 pounds will be reduced to 21,000 pounds. With this reduced weight and a power increase of 25 percent (an increase that would result if a motor as powerful as that in the medium truck were used), the speed on a 6-percent grade would be 18 miles per hour. A grade reduction from a 6- to a 4-percent grade would further increase the speed to 23 miles per hour. The combination of the three methods would thus increase the speed from 14 to 23 miles per hour. It is evident that the use of a combination of the three methods would result in a large cost to the interests concerned, and even then the speeds would not even approximate desirable road speeds.

DECELERATION TESTS MADE TO DETERMINE TRACTIVE RESISTANCE

The determination of tractive resistance is of secondary importance but it is a very necessary part of the study. Deceleration tests were made on each vehicle to determine the total tractive resistance, which is composed principally of the friction between tires and road surface, the inherent friction of the vehicle, and the resistance offered by the air. The first two forces named are commonly grouped as rolling resistance. The conversion of performance from one grade to another, the use of acceleration values to determine grade ability, the determination of efficiency factors, and the computation of a theoretical performance, all depend on this factor.

The deceleration of a vehicle when coasting on the level in neutral gear is proportional to the forces (rolling resistance and air resistance) that oppose the motion of the vehicle. The following equation expresses the relation:

$$TR = ma \qquad (4)$$

where

TR = total tractive resistance in pounds,
m = mass of vehicle, and
a = linear deceleration of vehicle in feet per second per second.

However, the above equation does not take into consideration a certain energy that is stored in the rotating parts when decelerating or accelerating. The energy of the rotating parts must be added to the energy of linear motion which is expressed by the above equation. The force equivalent to this energy is:

$$F = \frac{I}{r}\,\alpha \qquad (5)$$

where

F = force equivalent to energy of linear motion,
I = moment of inertia of rotating parts,
r = effective radius of rotating parts in inches, and
α = angular acceleration in radians per second per second.

When a vehicle is coasting in neutral, the only rotating parts decelerating are the wheels, brake drums, propeller shaft, and rear axle assembly. The moments of inertia of the propeller shaft and rear axle are so small in comparison to that for the wheels that it is practical to omit them from the consideration of the stored energy. For the wheels and brake drums the angular acceleration is equal to $\frac{a}{r}$ where a is equal to the linear deceleration and r is equal to the effective radius of the wheels. Substituting $\frac{a}{r}$ for α and combining the equations for the energy of the rotating parts and of linear motion the formula for determining tractive resistance is found to be:

$$TR = ma + \frac{I}{r^2}a \qquad (6)$$

or

$$= (m + k_0)\,a$$

where

TR = total tractive resistance in pounds,
m = mass of vehicle,
a = linear deceleration of vehicle in feet per second per second, and
k_0 = mass equivalent constant for neutral gear.

The mass equivalent constant can be determined experimentally if the deceleration is measured for a vehicle coasting on two different grades, one of which can be, and in this study was, level. As the total resistance on the grade is equal to the total resistance on the level for the same road speed and for the same load, the mass equivalent constant can be determined by solving the following equation for k_0:

$$W \sin A - a_g(m + k_0) = a_l(m + k_0) \qquad (7)$$

where

W = weight of vehicle in pounds,
A = angle in degrees that grade line makes with horizontal,
a_g = linear acceleration on grade in feet per second per second,
a_l = linear acceleration on level in feet per second per second,
m = mass of vehicle $= \frac{W}{32.2}$, and
k_0 = mass equivalent constant for neutral gear.

It can also be computed theoretically if the moments of inertia of the wheel assemblies are known. In most cases they were obtained from the manufacturer and used to compute a constant that could be used to check the experimental value. The theoretical k_0 is obtained by adding the moments of inertia for the wheels and brake drums and dividing the total by the effective radius squared. Table 7 contains the average mass equivalent constants, both theoretical and experimental, by capacity groups and vehicle types for all the vehicles tested.

TABLE 7.—*Average mass equivalent constants for neutral gear by capacity groups and vehicle types*

Capacity group and vehicle type	Tire size		Mass equivalent constants	
	Truck or tractor	Semi-trailer	Experimental value	Theoretical value
Light trucks	7.50×20		23	15
Light tractor-truck semitrailers	7.50×20	34×7	28	27
Medium trucks	9.00×20		29	25
Medium tractor-truck semitrailers	9.00×20	9.75×20	52	40
Heavy tractor-truck semitrailers	10.50×20	9.75×20	56	51

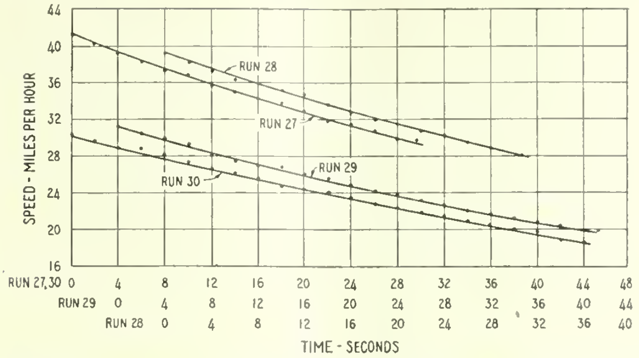

Figure 9.—Time-Speed Curves for Tractor-Truck Semitrailer Coasting on 0-Percent Grade With Gross Vehicle Weight of 12,000 Pounds.

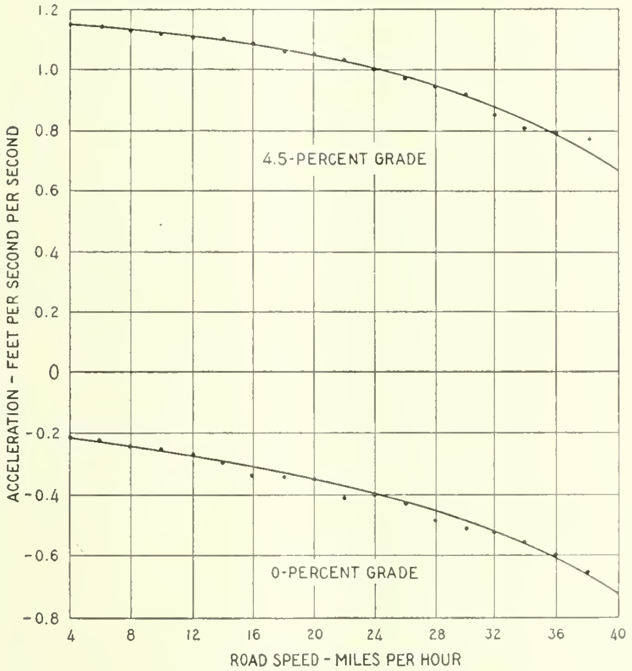

Figure 10.—Acceleration-Speed Curves for a Tractor-Truck Semitrailer Coasting on a 0-Percent and a 4.5-Percent Grade With a Gross Vehicle Weight of 12,000 Pounds.

The deceleration tests were made on two sections of concrete pavement. One was level and the other a 4½ percent grade. Values of deceleration were obtained for speeds ranging from 4 miles per hour to about 40 miles per hour. The effect of wind and any irregularities of grade on the tractive resistance was compensated for in some measure by test runs in both directions on the level section.

UNIT TRACTIVE RESISTANCE FOUND TO VARY WITH WEIGHT

At the start of the tests the tractive resistance was determined for but one gross weight. However, it was soon discovered that not only the total tractive resistance but also the unit resistance in pounds per thousand pounds varied appreciably with weight. Thereafter each vehicle was tested with three different loads. The difference in total tractive resistance for any two gross weights was proved to vary directly with the increase in weight, so it was possible to determine the tractive resistance for any combination of weight and speed.

A time-distance record of each deceleration run was obtained with the time-distance recorder described under the actual grade tests. The time-distance record was divided into 2-second intervals, and approximately instantaneous speeds were computed at each time interval. Time-speed curves were plotted with the time in seconds as the abscissa and the speed in miles per hour as the ordinate. Since the slope of the time-speed curve at any point is the deceleration in miles per hour per second, it is possible to determine the values of deceleration at any given speed. Figure 9 shows time-speed curves for one of the tractor-truck semitrailers.

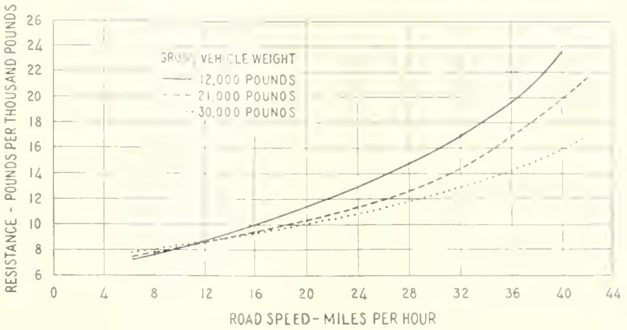

Figure 11.—Variation of Tractive Resistance With Speed and Weight for a Tractor-Truck Semitrailer.

The slope was measured at 2-mile-per-hour intervals by drawing tangents to the curve by means of a mirror that was specially developed for the purpose. The mirror is so silvered that it both reflects and transmits light. The mirror is held perpendicular to the plane of the curve and the bottom edge is placed over the point at which a determination is to be made. By causing the image of the part of the curve in front of the mirror to coincide with the part of the curve visible through the mirror, the vertical face of the mirror becomes a plane perpendicular to the tangent. Table 8 contains the values of deceleration obtained for one of the tractor-truck semitrailers having a gross weight of 12,000 pounds. The values of deceleration in table 8 are plotted against speed in figure 10. The decelerations used to compute the tractive resistance were transcribed from the smooth curve shown on this figure.

Table 8.—*Values of deceleration for a tractor-truck semitrailer coasting on a 0-percent grade with a gross vehicle weight of 12,000 pounds*

Speed, m. p. h.	Deceleration in miles per hour per second for —									
	Run No. 27, north	Run No. 28, south	Run No. 29, north	Run No. 30, south	Run No. 31, north	Run No. 32, south	Run No. 33, north	Run No. 34, south	Average	
									M. p. h./sec.	*Ft./sec./sec.*
2							0. 145	0. 141	0. 143	0. 210
4							. 154	. 141	. 148	. 217
6							. 162	. 141	. 152	. 223
8							. 173	. 151	. 162	. 238
10					0. 172	0. 163	184	. 158	. 169	. 248
12					. 182	. 177	. 197	. 181	. 184	. 270
14					. 194	. 181	. 215	. 210	. 200	. 293
16					. 213	. 197	. 241	. 214	. 224	. 329
18					. 237	. 218			. 228	. 334
20			0. 234	0. 241	. 270	. 246			. 248	. 364
22			. 257	. 253	. 306	. 291			. 277	. 406
24			. 271	. 271					. 271	. 397
26			. 299	. 294					. 296	. 434
28			. 332	. 322					. 327	. 480
30	0. 359	0. 320	. 378	. 348					. 351	. 515
32	. 374	. 341							. 358	. 525
34	. 393	. 371							. 382	. 560
36	. 414	. 404							. 409	. 600
38	. 444	. 456							. 450	. 660

The total tractive resistance that was computed from the deceleration shown in figure 10 for the 0-percent grade is contained in table 9. The total resistance is divided by the thousands of pounds of gross weight (12) to determine the coefficient of tractive resistance in pounds per 1,000 pounds. The vehicle for which these results were derived was also tested with weights of 21,000 and 30,000 pounds. The coefficients of tractive resistance for each of the three weights are shown in figure 11 to indicate how the tractive resistance varies with gross weight as well as with speed.

TABLE 9.—*Tractive resistance*[1] *for a tractor-truck semitrailer with a gross vehicle weight of 12,000 pounds*

Speed, m. p. h.	Deceleration	Total tractive resistance	Unit tractive resistance	Speed, m. p. h.	Deceleration	Total tractive resistance	Unit tractive resistance
	Ft./sec./sec.	*Pounds*	*Lb./1,000 lb.*		*Ft./sec./sec.*	*Pounds*	*Lb./1,000 lb.*
4	0.212	82.5	6.9	24	0.401	156.1	13.0
6	.223	86.7	7.2	26	.430	167.4	13.9
8	.239	93.0	7.7	28	.459	178.2	14.8
10	.253	98.4	8.2	30	.490	190.8	15.9
12	.271	105.4	8.8	32	.523	203.3	16.9
14	.290	112.8	9.4	34	.561	218.2	18.2
16	.310	120.6	10.0	36	.604	234.8	19.6
18	.330	128.4	10.7	38	.660	256.8	21.4
20	.352	137.0	11.4	40	.728	283.0	23.6
22	.378	147.0	12.2				

[1] Tractive resistance, $TR=-a(m+k_0)$
$m=373$
$k_0=16$
$TR=-389a$

From the total resistance in pounds determined for the three weights, tables are prepared that enable the tractive resistance to be obtained for any weight and speed within the limits of the tests. Since the difference in total resistance for any two gross weights was proved to vary directly with the increase in weight, it was possible to prorate the difference to other weights. Table 10 shows the total tractive resistance for various loads and speeds and table 11 lists the coefficients that were determined from the values of total tractive resistance given in table 10. Similar results were developed for each of the vehicles tested.

It was necessary to know the tractive resistance for each of the vehicles tested in order that certain analysis could be accurately made. However, in order to obtain values of unit tractive resistance that are more applicable for general use, the average tractive resistance has been obtained from the results of the individual vehicles for the single unit trucks and the tractor-truck semitrailers in each size group (light, medium, and heavy).

TABLE 11.—*Unit tractive resistance for a tractor-truck semitrailer at various speeds and weights*

Speed, m. p. h.	Unit tractive resistance, in pounds per 1,000 pounds, for weights of—										
	12,000 pounds	14,000 pounds	16,000 pounds	18,000 pounds	20,000 pounds	21,000 pounds	23,000 pounds	25,000 pounds	27,000 pounds	29,000 pounds	30,000 pounds
6	7.2	7.2	7.3	7.3	7.3	7.3	7.4	7.5	7.6	7.7	7.8
8	7.7	7.8	7.8	7.8	7.8	7.8	7.9	7.9	8.0	8.0	8.1
10	8.2	8.2	8.2	8.2	8.2	8.2	8.3	8.3	8.3	8.4	8.4
12	8.8	8.8	8.7	8.7	8.7	8.7	8.7	8.7	8.7	8.7	8.7
14	9.4	9.3	9.2	9.2	9.2	9.2	9.1	9.1	9.1	9.0	9.0
16	10.0	9.9	9.7	9.6	9.6	9.5	9.5	9.5	9.4	9.4	9.4
18	10.7	10.4	10.2	10.1	9.9	9.9	9.8	9.8	9.8	9.8	9.8
20	11.4	11.1	10.8	10.6	10.5	10.4	10.3	10.3	10.2	10.1	10.1
22	12.2	11.8	11.4	11.1	10.9	10.8	10.7	10.7	10.6	10.6	10.5
24	13.0	12.5	12.1	11.8	11.5	11.4	11.3	11.2	11.2	11.1	11.1
26	13.9	13.3	12.8	12.4	12.1	12.0	11.8	11.7	11.6	11.5	11.5
28	14.8	14.1	13.6	13.2	12.9	12.7	12.5	12.3	12.1	12.0	11.9
30	15.9	15.1	14.5	14.1	13.7	13.5	13.2	12.9	12.7	12.5	12.4
32	16.9	16.1	15.5	15.0	14.6	14.5	14.0	13.7	13.4	13.1	13.0
34	18.2	17.3	16.7	16.2	15.8	15.6	15.0	14.6	14.2	13.8	13.6
36	19.6	18.7	18.0	17.5	17.1	16.9	16.1	15.5	14.9	14.5	14.3
38	21.4	20.4	19.7	19.1	18.6	18.4	17.5	16.7	16.0	15.4	15.2
40	23.6	22.4	21.5	20.8	20.3	20.0	18.9	17.9	17.1	16.4	16.1
42						21.6	20.3	19.2	18.2	17.3	17.0

The average unit tractive resistance in pounds per thousand pounds for the light and medium single unit trucks and for the light, medium, and heavy tractor-truck semitrailers is shown in tables 12 to 16 inclusive. These tables contain the coefficient of tractive resistance for various conditions of weight and speed. For the light tractor-truck semitrailers, table 14, the unit tractive resistance is shown for speeds ranging from 4 to 38 miles per hour and for gross weights ranging from 12,000 to 30,000 pounds. At the lower speeds the variation of unit tractive resistance with weight is negligible. At 4 miles per hour the coefficient is 7.2 at 12,000 pounds and 7.8 at 30,000 pounds. However, at the higher speeds there is a large variation. At 38 miles per hour the coefficient is 23.7 for 12,000 pounds and only 15.3 for 30,000 pounds, a decrease of 35 percent. The results have been confined to the range of loads and speeds used on the tests, but the values can be expanded to determine the approximate resistance for speeds and weights not included in these results.

The average unit tractive resistance for the light trucks is compared with that for the light tractor-truck semitrailers in figure 12 for a common weight of 16,000

TABLE 10.—*Total tractive resistance for a tractor-truck semitrailer at various speeds and weights*

Speed m. p. h.	Total tractive resistance for weights of—											Increase, 12,000 to 21,000 pounds		Increase, 21,000 to 30,000 pounds	
	12,000 pounds	14,000 pounds	16,000 pounds	18,000 pounds	20,000 pounds	21,000 pounds	23,000 pounds	25,000 pounds	27,000 pounds	29,000 pounds	30,000 pounds	Total	Per 2,000 pounds	Total	Per 2,000 pounds
6	86.7	101.4	116.1	130.8	145.5	153.0	170.8	188.6	206.4	224.2	233.1	66.3	14.7	80.1	17.8
8	93.0	108.7	124.4	140.1	155.8	163.8	181.2	198.6	216.0	233.4	242.0	70.8	15.7	78.2	17.4
10	98.4	115.0	131.6	148.2	164.8	173.0	190.3	207.6	224.6	242.2	251.3	74.6	16.6	78.3	17.4
12	105.4	122.5	139.6	156.7	173.8	182.3	199.5	216.7	233.9	251.1	259.9	76.9	17.1	77.6	17.2
14	112.8	130.4	148.0	165.6	183.2	192.2	209.6	227.0	244.4	261.8	270.4	79.4	17.6	78.2	17.4
16	120.6	138.2	155.8	173.4	191.0	199.7	218.0	236.3	254.6	272.9	282.2	79.1	17.6	82.5	18.3
18	128.4	145.9	163.4	180.9	198.4	207.0	226.1	245.2	264.3	283.4	292.9	78.6	17.5	85.9	19.1
20	137.0	155.0	173.0	191.0	209.0	217.9	237.0	256.1	275.2	294.3	304.0	80.9	18.0	86.1	19.1
22	147.0	164.8	182.6	200.4	218.2	227.0	246.9	266.8	286.7	306.6	316.5	80.0	17.8	89.5	19.9
24	156.1	174.7	193.3	211.9	230.5	239.9	260.4	280.9	301.4	321.9	332.0	83.8	18.6	92.1	20.5
26	167.4	186.0	204.6	223.2	241.8	251.0	271.5	292.0	312.5	333.0	343.4	83.6	18.6	92.4	20.5
28	178.2	197.9	217.6	237.3	257.0	267.1	287.2	307.3	327.4	347.5	357.5	88.9	19.7	90.4	20.1
30	190.8	211.5	232.2	252.9	273.6	284.0	303.5	323.0	342.5	362.0	372.0	93.2	20.7	88.0	19.5
32	203.3	225.7	248.1	270.5	292.9	304.0	323.1	342.2	361.3	380.4	390.0	100.7	22.4	86.0	19.1
34	218.2	242.4	266.6	290.8	315.0	327.0	345.4	363.8	382.2	400.6	409.9	108.8	24.2	82.9	18.4
36	234.8	261.4	288.0	314.6	341.2	354.9	371.1	387.3	403.5	419.7	428.0	120.1	26.7	73.1	16.2
38	256.8	285.8	314.8	343.8	372.8	387.1	402.2	417.3	432.4	447.5	455.0	130.3	29.0	67.9	15.1
40	283.0	313.6	344.2	374.9	405.4	421.0	434.6	448.2	461.8	475.4	482.0	138.0	30.6	61.0	13.6
42						455.0	467.0	479.0	491.0	503.0	509.0			54.0	12.0

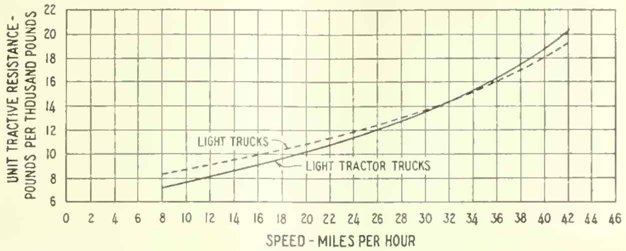

Figure 12.—Variation of Unit Tractive Resistance With Speed for Light Trucks and Light Tractor-Truck Semitrailers With a 16,000-Pound Gross Weight.

pounds. The variation in unit resistance shown by this comparison is slight, as would be expected, since the frontal area is about the same for the two types of vehicles. The addition of a third axle has little effect on the unit tractive resistance.

The average unit tractive resistance for the light, medium, and heavy tractor-truck semitrailers is shown in figure 13 for a common weight of 16,000 pounds, to-

Table 12.—*Average unit tractive resistance for light trucks (6 makes)*

Speed, m. p. h.	Unit tractive resistance, in pounds per 1,000 pounds, for weights of—								
	8,000 pounds	9,000 pounds	10,000 pounds	11,000 pounds	12,000 pounds	13,000 pounds	14,000 pounds	15,000 pounds	16,000 pounds
4	8.3	8.3	8.4	8.4	8.4	8.5	8.5	8.5	8.6
6	8.8	8.8	8.8	8.8	8.8	8.8	8.9	8.9	8.9
8	9.3	9.3	9.3	9.3	9.3	9.2	9.2	9.2	9.2
10	10.0	9.9	9.8	9.8	9.7	9.7	9.7	9.6	9.6
12	10.7	10.6	10.5	10.4	10.3	10.2	10.1	10.0	10.0
14	11.6	11.4	11.2	11.0	10.8	10.7	10.6	10.5	10.4
16	12.5	12.2	11.9	11.7	11.5	11.3	11.1	11.0	10.9
18	13.6	13.1	12.7	12.4	12.2	11.9	11.7	11.5	11.4
20	14.7	14.1	13.6	13.2	12.9	12.6	12.3	12.1	11.9
22	15.9	15.2	14.6	14.1	13.7	13.3	13.0	12.7	12.5
24	17.2	16.3	15.6	15.0	14.5	14.1	13.7	13.4	13.1
26	18.5	17.5	16.7	16.0	15.4	14.9	14.5	14.1	13.8
28	19.9	18.8	17.8	17.1	16.4	15.9	15.4	14.9	14.6
30	21.4	20.1	19.0	18.2	17.4	16.8	16.3	15.8	15.4
32	22.9	21.4	20.3	19.3	18.5	17.8	17.2	16.7	16.2
34	24.5	22.9	21.6	20.5	19.7	18.9	18.2	17.6	17.1
36	26.1	24.4	23.0	21.9	20.9	20.1	19.3	18.6	18.1
38	27.9	26.1	24.6	23.3	22.3	21.4	20.5	19.8	19.2
40	29.9	27.9	26.3	25.0	23.9	22.8	21.9	21.1	20.4

Table 13.—*Average unit tractive resistance for medium trucks (3 makes)*

Speed, m. p. h.	Unit tractive resistance, in pounds per 1,000 pounds, for weights of—													
	11,000 pounds	12,000 pounds	13,000 pounds	14,000 pounds	15,000 pounds	16,000 pounds	17,000 pounds	18,000 pounds	19,000 pounds	20,000 pounds	21,000 pounds	22,000 pounds	23,000 pounds	24,000 pounds
4	8.7	8.7	8.6	8.6	8.6	8.5	8.5	8.7	8.9	9.1	9.3	9.4	9.5	9.6
6	9.4	9.3	9.2	9.1	9.1	9.0	9.0	9.1	9.3	9.4	9.5	9.7	9.7	9.8
8	10.0	9.9	9.8	9.7	9.6	9.6	9.5	9.6	9.7	9.8	9.9	9.9	10.0	10.0
10	10.9	10.7	10.5	10.3	10.2	10.1	10.0	10.1	10.1	10.1	10.2	10.2	10.2	10.3
12	11.6	11.4	11.2	11.0	10.8	10.7	10.6	10.6	10.5	10.6	10.6	10.6	10.6	10.6
14	12.6	12.4	11.9	11.7	11.5	11.3	11.1	11.1	11.2	11.1	11.1	11.1	11.0	11.0
16	13.5	13.1	12.8	12.5	12.2	12.0	11.8	11.7	11.7	11.6	11.6	11.6	11.5	11.5
18	14.6	14.1	13.7	13.3	12.9	12.7	12.4	12.4	12.3	12.2	12.2	12.1	12.1	12.0
20	15.7	15.1	14.6	14.1	13.8	13.4	13.1	13.0	13.0	12.9	12.8	12.8	12.7	12.6
22	16.9	16.2	15.6	15.1	14.6	14.2	13.9	13.8	13.7	13.6	13.5	13.4	13.4	13.3
24	18.1	17.3	16.6	16.0	15.5	15.1	14.7	14.5	14.4	14.3	14.2	14.1	14.1	14.0
26	19.4	18.5	17.7	17.0	16.5	15.9	15.5	15.4	15.2	15.1	15.0	14.9	14.8	14.7
28	20.8	19.8	18.9	18.2	17.5	16.9	16.5	16.3	16.1	16.0	15.9	15.8	15.6	15.5
30	22.5	21.4	20.4	19.5	18.8	18.1	17.5	17.3	17.2	17.0	16.9	16.7	16.6	16.5
32	24.1	22.9	21.8	20.9	20.1	19.4	18.7	18.5	18.3	18.1	17.9	17.7	17.6	17.4
34	26.1	24.7	23.5	22.5	21.7	20.9	20.2	19.9	19.6	19.3	19.1	18.8	18.6	18.5
36	28.5	26.9	25.6	24.5	23.5	22.6	21.9	21.5	21.1	20.8	20.5	20.2	20.0	19.7
38	31.6	29.9	28.4	27.2	26.1	25.1	24.3	23.7	23.2	22.7	22.3	21.9	21.6	21.3

Table 14.—*Average unit tractive resistance for light tractor trucks (7 makes)*

Speed, m. p. h.	Unit tractive resistance, in pounds per 1,000 pounds, for weights of—									
	12,000 pounds	14,000 pounds	16,000 pounds	18,000 pounds	20,000 pounds	22,000 pounds	24,000 pounds	26,000 pounds	28,000 pounds	30,000 pounds
4	7.2	7.3	7.3	7.4	7.5	7.6	7.6	7.7	7.8	7.8
6	7.6	7.7	7.7	7.7	7.7	7.8	7.9	7.9	8.0	8.0
8	8.2	8.2	8.1	8.1	8.1	8.2	8.2	8.3	8.3	8.3
10	8.8	8.7	8.6	8.6	8.5	8.6	8.6	8.6	8.6	8.7
12	9.4	9.3	9.1	9.0	9.0	9.0	9.0	9.0	9.0	9.0
14	10.1	9.8	9.7	9.5	9.4	9.4	9.3	9.3	9.3	9.3
16	10.8	10.5	10.2	10.0	9.9	9.8	9.7	9.7	9.7	9.6
18	11.5	11.1	10.8	10.5	10.3	10.2	10.1	10.1	10.0	10.0
20	12.3	11.8	11.4	11.1	10.9	10.7	10.6	10.5	10.4	10.4
22	13.2	12.6	12.1	11.7	11.4	11.2	11.1	11.0	10.9	10.8
24	14.2	13.4	12.8	12.4	12.0	11.8	11.6	11.5	11.3	11.2
26	15.2	14.3	13.6	13.1	12.7	12.4	12.2	12.0	11.8	11.7
28	16.3	15.2	14.5	13.9	13.4	13.0	12.8	12.5	12.3	12.2
30	17.4	16.3	15.4	14.7	14.2	13.7	13.4	13.1	12.9	12.7
32	18.7	17.4	16.4	15.6	15.0	14.5	14.1	13.8	13.5	13.2
34	20.1	18.7	17.6	16.7	16.0	15.4	14.9	14.5	14.2	13.8
36	21.8	20.1	18.9	17.9	17.1	16.4	15.8	15.3	14.9	14.5
38	23.7	21.8	20.4	19.3	18.5	17.6	16.9	16.3	15.8	15.3

Table 15.—*Average unit tractive resistance for medium tractor trucks (6 makes)*

Speed, m. p. h.	Unit tractive resistance, in pounds per 1000 pounds, for weights of—												
	14,000 pounds	16,000 pounds	18,000 pounds	20,000 pounds	22,000 pounds	24,000 pounds	26,000 pounds	28,000 pounds	30,000 pounds	32,000 pounds	34,000 pounds	36,000 pounds	38,000 pounds
6	7.1	7.1	7.1	7.1	7.1	7.1	7.1	7.2	7.3	7.4	7.4	7.5	7.5
8	7.7	7.7	7.6	7.6	7.5	7.5	7.5	7.5	7.6	7.7	7.7	7.8	7.8
10	8.4	8.2	8.1	8.0	7.9	7.9	7.8	7.9	7.9	7.9	8.0	8.0	8.0
12	9.1	8.8	8.6	8.5	8.4	8.3	8.2	8.2	8.2	8.3	8.3	8.3	8.3
14	9.8	9.4	9.2	9.0	8.8	8.7	8.6	8.6	8.6	8.6	8.6	8.6	8.6
16	10.5	10.1	9.8	9.5	9.3	9.1	9.0	9.0	9.0	8.9	8.9	8.9	8.9
18	11.2	10.8	10.4	10.1	9.8	9.6	9.5	9.4	9.4	9.3	9.3	9.3	9.2
20	12.0	11.4	11.0	10.7	10.4	10.1	9.9	9.8	9.8	9.7	9.6	9.6	9.6
22	12.9	12.2	11.7	11.3	10.9	10.6	10.4	10.3	10.2	10.1	10.0	10.0	9.9
24	13.8	13.0	12.4	11.9	11.5	11.2	10.9	10.7	10.6	10.5	10.4	10.4	10.3
26	14.8	13.9	13.2	12.6	12.1	11.8	11.4	11.3	11.1	11.0	10.9	10.8	10.7
28	15.8	14.8	14.0	13.4	12.8	12.4	12.0	11.8	11.7	11.5	11.4	11.2	11.1
30	17.0	15.8	14.9	14.2	13.6	13.1	12.7	12.4	12.2	12.0	11.9	11.7	11.6
32	18.3	17.0	16.0	15.2	14.5	13.9	13.4	13.1	12.9	12.7	12.4	12.3	12.1
34	19.9	18.3	17.2	16.2	15.5	14.8	14.3	13.9	13.6	13.3	13.1	12.9	12.7
36	21.4	19.8	18.5	17.4	16.5	15.8	15.2	14.8	14.4	14.1	13.8	13.6	13.3
38	23.4	21.5	20.0	18.8	17.9	17.1	16.4	15.9	15.4	15.0	14.7	14.4	14.1
40	25.9	23.7	22.0	20.6	19.5	18.6	17.8	17.2	16.6	16.1	15.7	15.3	14.9

Table 16.—*Average unit tractive resistance for heavy tractor trucks (4 makes)*

Speed, m. p. h.	Unit tractive resistance, in pounds per 1,000 pounds, for weights of—												
	16,000 pounds	18,000 pounds	20,000 pounds	22,000 pounds	24,000 pounds	26,000 pounds	28,000 pounds	30,000 pounds	32,000 pounds	34,000 pounds	36,000 pounds	38,000 pounds	40,000 pounds
6	7.9	7.8	7.7	7.6	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5
8	8.5	8.3	8.2	8.1	8.0	8.0	8.0	7.9	7.9	7.9	7.8	7.8	7.8
10	9.2	9.0	8.8	8.6	8.5	8.4	8.4	8.3	8.3	8.2	8.2	8.1	8.1
12	10.0	9.7	9.5	9.3	9.1	9.0	8.9	8.8	8.7	8.7	8.6	8.5	8.5
14	10.8	10.4	10.1	9.9	9.7	9.5	9.4	9.3	9.2	9.1	9.0	8.9	8.9
16	11.6	11.1	10.8	10.5	10.3	10.1	9.9	9.8	9.6	9.5	9.4	9.3	9.2
18	12.4	11.9	11.5	11.2	10.9	10.7	10.5	10.3	10.1	10.0	9.8	9.7	9.6
20	13.3	12.7	12.3	11.9	11.6	11.3	11.1	10.8	10.6	10.5	10.3	10.2	10.1
22	14.2	13.6	13.1	12.6	12.3	11.9	11.6	11.4	11.2	11.0	10.8	10.6	10.5
24	15.2	14.5	13.9	13.4	13.0	12.6	12.3	12.0	11.7	11.5	11.3	11.1	10.9
26	16.2	15.4	14.7	14.2	13.7	13.3	12.9	12.6	12.3	12.0	11.8	11.6	11.4
28	17.2	16.3	15.6	15.0	14.5	14.0	13.6	13.2	12.9	12.6	12.3	12.1	11.9
30	18.3	17.3	16.5	15.8	15.3	14.7	14.3	13.9	13.5	13.2	12.9	12.7	12.5
32	19.4	18.3	17.4	16.7	16.1	15.5	15.0	14.6	14.2	13.9	13.6	13.3	13.0
34	20.5	19.4	18.4	17.7	17.0	16.4	15.8	15.4	15.0	14.6	14.3	14.0	13.7
36	21.8	20.5	19.5	18.7	18.0	17.4	16.8	16.3	15.8	15.5	15.1	14.8	14.5
38	23.1	21.8	20.7	19.9	19.1	18.4	17.8	17.3	16.8	16.4	16.0	15.7	15.4
40	24.6	23.2	22.1	21.1	20.4	19.6	19.0	18.4	17.9	17.5	17.1	16.7	16.4

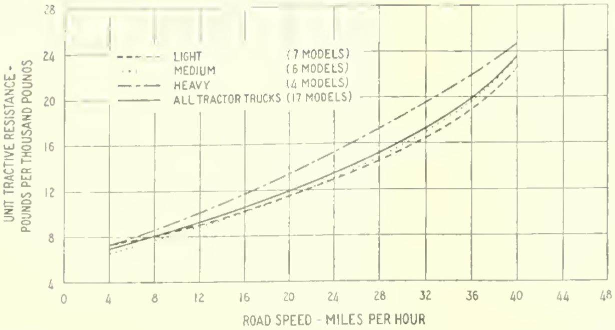

Figure 13.—Variation of Unit Tractive Resistance With Speed for All Tractor-Truck Semitrailers With a 16,000-Pound Gross Weight.

gether with the average unit resistance for all the vehicles of this type. The average tractive resistances indicated by the four curves are in close agreement with the exception of those shown for the heavy tractor trucks. Only four heavy tractor trucks were tested and one of these had a very high tractive resistance. The average resistance of the other three heavy vehicles is in line with that shown for the light and medium capacity groups.

AVERAGE VALUES OF UNIT TRACTIVE RESISTANCE ACCEPTABLE FOR ALL PRACTICAL PURPOSES

The comparisons made in figures 12 and 13 prove that there is little variation in the average unit tractive resistance for the types and sizes of vehicles considered in this study. For this reason it is acceptable for all practical purposes to use average values of tractive resistance for all the single units and other average values for all the combination units. These average values of the unit tractive resistance are given in tables 17 and 18. For the single unit trucks (table 17) the resistance is shown for weights ranging from 8,000 to 24,000 pounds and for speeds ranging from 4 to 38 miles per hour. For the tractor-truck semitrailers (table 18) it is shown for weights ranging from 12,000 to 42,000 pounds and for speeds ranging from 6 to 40 miles per hour. There is little variation in the unit tractive resistance for a given speed and weight between the average values obtained for the two types of vehicles as is shown in figure 14.

Table 17.—*Average unit tractive resistance for all single-unit trucks*

Speed, m. p. h.	Unit tractive resistance, in pounds per 1,000 pounds, for weights of—								
	8,000 pounds	10,000 pounds	12,000 pounds	14,000 pounds	16,000 pounds	18,000 pounds	20,000 pounds	22,000 pounds	24,000 pounds
4	8.4	8.5	8.5	8.6	8.6	8.6	8.6	8.6	8.6
6	9.0	9.0	9.0	9.0	8.9	8.9	8.9	8.9	8.9
8	9.8	9.6	9.5	9.4	9.3	9.3	9.2	9.2	9.2
10	10.6	10.3	10.0	9.9	9.8	9.7	9.6	9.5	9.5
12	11.5	11.0	10.6	10.4	10.2	10.1	10.0	9.9	9.8
14	12.4	11.7	11.3	11.0	10.7	10.5	10.4	10.2	10.1
16	13.5	12.6	12.0	11.6	11.2	11.0	10.8	10.6	10.5
18	14.8	13.6	12.8	12.2	11.8	11.5	11.2	11.0	10.8
20	16.0	14.6	13.6	12.9	12.4	12.0	11.7	11.4	11.2
22	17.4	15.7	14.5	13.7	13.1	12.6	12.2	11.9	11.6
24	18.8	16.8	15.4	14.5	13.8	13.2	12.8	12.4	12.1
26	20.3	18.0	16.5	15.3	14.5	13.9	13.4	12.9	12.6
28	21.9	19.3	17.5	16.3	15.4	14.6	14.0	13.6	13.2
30	23.6	20.6	18.7	17.3	16.3	15.5	14.8	14.3	13.9
32	25.3	22.1	19.9	18.4	17.3	16.4	15.7	15.1	14.6
34	27.2	23.7	21.3	19.6	18.4	17.4	16.6	16.0	15.4
36	29.5	25.5	22.9	21.0	19.6	18.5	17.6	16.9	16.3
38	32.0	27.7	24.8	22.7	21.2	20.0	19.0	18.2	17.5

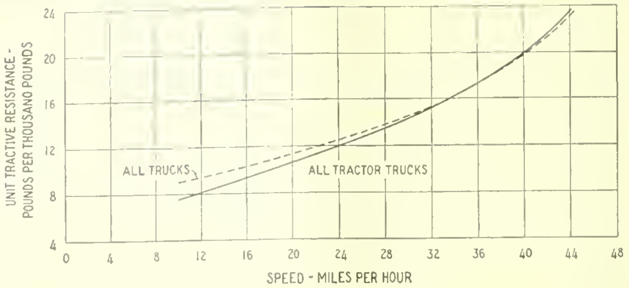

Figure 14.—Variation of Unit Tractive Resistance With Speed for All Trucks and Tractor-Truck Semitrailers With a 16,000-Pound Gross Weight.

The average deviations of the unit tractive resistance for the single-unit trucks and for the tractor-truck semitrailers from the average values shown in tables 18 and 19 are 10.0 and 11.3 percent, respectively. Assuming that the average deviation is about 10 percent, which appears to be a reasonable value, the error that might be introduced in the computation of grade ability with the average values for a given vehicle would not be great. For example, for a gross weight of 40,000 pounds and a speed of 20 miles per hour, the coefficient of tractive resistance is 9.8 pounds per 1,000 pounds. The average expectant error would be 1 pound per 1,000 pounds, which is equivalent to a 0.1 percent grade. If a 4-percent grade was being considered, the resultant error would be only 2 percent.

An important point to remember when using the tractive resistances reported here is that the tests were made on trucks equipped with a body 8 feet wide with 24-inch sideboards, and on tractor-trucks with a semitrailer 8 feet wide with 18-inch sideboards. If a van type body had been used instead of the platform body, the tractive resistance would undoubtedly have been greater than that shown, particularly for the higher speeds. Also, since all tests were made on concrete surfaces, the application of the results must be limited accordingly.

The development of a method for computing the performance of a vehicle from its specified characteristics required a determination of efficiency as well as tractive resistance. The common practice has been to assume an efficiency factor in order to allow for losses in the transfer of power from the clutch to the driving wheels. However, no efficiency factor has yet received general acceptance. The results of the actual grade tests, together with the certified power, have been used to provide efficiency factors for all the vehicles tested. The efficiency factors were obtained by applying the results of the actual grade tests to a basic performance formula derived by equating the force produced at the driving wheels to the sum of the grade resistance and the tractive resistance.

The basic performance formula used to compute the grade ability of a vehicle is as follows:

$$GVW=\frac{T\times GR\times E}{r(f+g)}\text{ ------------ }(8)$$

where

GVW = gross vehicle weight in pounds,
T = torque at a given engine speed in pound-inches,
GR = total gear reduction,
E = efficiency,

TABLE 18.—*Average unit tractive resistance for all tractor trucks*

Speed, m. p. h.	Unit tractive resistance, in pounds per 1,000 pounds, for weights of—															
	12,000 pounds	14,000 pounds	16,000 pounds	18,000 pounds	20,000 pounds	22,000 pounds	24,000 pounds	26,000 pounds	28,000 pounds	30,000 pounds	32,000 pounds	34,000 pounds	36,000 pounds	38,000 pounds	40,000 pounds	42,000 pounds
6	7.5	7.5	7.5	7.6	7.6	7.6	7.6	7.6	7.6	7.6	7.6	7.7	7.7	7.7	7.7	7.7
8	8.1	8.1	8.1	8.0	8.0	8.0	8.0	8.0	8.0	8.0	8.0	8.0	8.0	8.0	8.0	7.9
10	8.8	8.7	8.6	8.5	8.5	8.4	8.4	8.4	8.3	8.3	8.3	8.3	8.2	8.2	8.2	8.2
12	9.6	9.4	9.2	9.1	9.0	8.9	8.8	8.8	8.7	8.7	8.6	8.6	8.5	8.5	8.5	8.5
14	10.4	10.1	9.8	9.6	9.5	9.4	9.3	9.2	9.1	9.0	9.0	8.9	8.9	8.8	8.8	8.8
16	11.2	10.8	10.5	10.2	10.0	9.8	9.7	9.6	9.5	9.4	9.3	9.3	9.2	9.2	9.1	9.1
18	12.1	11.6	11.2	10.8	10.6	10.4	10.2	10.1	9.9	9.8	9.7	9.6	9.6	9.5	9.4	9.4
20	13.0	12.4	11.9	11.5	11.2	10.9	10.7	10.5	10.4	10.3	10.1	10.0	9.9	9.9	9.8	9.7
22	14.0	13.2	12.6	12.2	11.8	11.5	11.3	11.0	10.9	10.7	10.6	10.4	10.3	10.2	10.2	10.1
24	15.4	14.1	13.4	12.9	12.5	12.1	11.8	11.6	11.4	11.2	11.0	10.9	10.7	10.6	10.5	10.4
26	16.2	15.1	14.3	13.7	13.2	12.8	12.4	12.1	11.9	11.7	11.5	11.3	11.2	11.1	10.9	10.8
28	17.4	16.2	15.2	14.5	13.9	13.5	13.1	12.8	12.5	12.2	12.0	11.8	11.6	11.5	11.4	11.2
30	18.7	17.3	16.2	15.4	14.8	14.2	13.8	13.4	13.1	12.8	12.5	12.3	12.1	12.0	11.8	11.7
32	20.1	18.5	17.3	16.4	15.6	15.0	14.5	14.1	13.8	13.4	13.2	12.9	12.7	12.5	12.3	12.2
34	21.7	19.9	18.5	17.5	16.6	16.0	15.4	14.8	14.5	14.1	13.8	13.5	13.3	13.1	12.9	12.7
36	23.4	21.4	19.9	18.7	17.8	17.0	16.3	15.8	15.3	14.9	14.6	14.3	14.0	13.7	13.5	13.3
38	25.5	23.2	21.5	20.1	19.0	18.2	17.4	16.8	16.3	15.8	15.4	15.1	14.8	14.5	14.2	14.0
40	27.9	25.2	23.3	21.8	20.6	19.6	18.7	18.0	17.4	16.9	16.4	16.0	15.7	15.3	15.1	14.8

TABLE 19.—*The determination of over-all efficiency for a typical tractor-truck semitrailer*

[Make—A. Model—X. Transmission gear—2d. Total gear reduction—20.58. Grade—4½ percent]

Corrected *GVW*	Sustained speed	Rolling radius	Engine speed	Manufacturer's engine torque		Coefficient of tractive resistance	$f+g$	Over-all efficiency	
				Maximum	Net			Maximum	Net
Pounds	*M. p. h.*	*Inches*	*R. p. m.*	*Lb.-ft.*	*Lb.-ft.*	*Lb./lb.*	*Lb./lb.*	*Percent*	*Percent*
33,800	12.0	16.83	2,465	154	144	0.0087	0.0537	80.4	86.1
33,800	12.0	16.83	2,465	154	144	.0087	.0537	80.4	86.1
33,100	12.5	16.77	2,580	151	141	.0088	.0538	80.2	85.8
33,100	12.6	16.77	2,600	151	140	.0088	.0538	80.2	86.4
31,600	13.4	16.96	2,730	148	136	.0089	.0539	79.3	86.2
31,600	13.4	16.96	2,730	148	136	.0089	.0539	79.3	86.2
29,600	14.9	17.03	3,025	139	125	.0092	.0542	79.7	88.7
29,600	14.7	17.03	2,980	140	127	.0091	.0541	79.2	87.3
27,500	16.0	17.17	3,220	130	116	.0094	.0544	80.0	89.7
27,500	15.7	17.17	3,160	133	119	.0094	.0544	78.2	87.3
27,600	15.6	17.17	3,140	134	120	.0093	.0543	77.9	86.9
24,400	16.8	17.22	3,340	124	109	.0096	.0546	74.9	85.3

r=effective radius of driving wheels in inches,
f=coefficient of tractive resistance in pounds per pound of weight, and
g=grade in feet of rise per foot.

The efficiency is then determined by solving equation 8 for E as follows:

$$E=\frac{GVW(f+g)r}{T\times GR} \qquad (9)$$

where the numerator represents the torque actually produced at the driving wheels and the denominator the torque that would have been produced at the same point if there were no losses during the transmission of power.

EFFICIENCY COMPUTED FOR A SAMPLE TRUCK

The actual grade tests measured the speed that a vehicle could maintain on a given grade with a given gross vehicle weight. The coefficient of tractive resistance was determined by the deceleration tests that have been described. The effective radius of the driving wheels was measured in the field for each gross vehicle weight. The torque and the total gear reduction were obtained from specifications supplied by the manufacturer. All information necessary for determination of the efficiency was therefore available.

For example, a certain tractor-truck semitrailer maintained a uniform speed of 25.2 miles per hour in fourth gear on a 4.5-percent grade with a weight of 12,500 pounds. The radius of the driving wheels for this weight and the coefficient of tractive resistance for the weight and speed involved were determined to be 16.9 inches and 0.015 pounds per pound of weight, respectively. The total gear reduction when the vehicle was operated in fourth gear was 7.15.

Charts that show the certified power and torque at various engine speeds were used to obtain the torque produced by the engine. In order to obtain the torque it was first necessary to find the engine speed equivalent to the road speed of 25.2 miles per hour. The engine speed was computed in the following manner:

$$RPM=\frac{168\times GR\times S}{r} \qquad (10)$$

$$=\frac{168\times 7.15\times 25.2}{16.9}=1{,}780,$$

where

RPM=engine speed in revolutions per minute,
S=road speed in miles per hour,
GR=total gear reduction,
r=effective radius of driving wheels in inches, and
168=factor to convert units to revolutions per minute.

The torque produced at 1,780 revolutions per minute was 165 and 156 pound-feet, the manufacturer's certified maximum and net torque respectively. The efficiency factors for the maximum and net torque were then determined to be 90.5 percent and 94.4 percent by substituting the values given above in the equation for efficiency:

1. For maximum torque,

$$E=\frac{12500\ (0.015+0.045)\ 16.9}{165\times 12\times 7.15}=90.5.$$

2. For net torque,

$$E=\frac{12500\ (0.015+0.045)\ 16.9}{156\times 12\times 7.15}=94.4.$$

The maximum torque is produced by an engine that is stripped of all accessories except those that are necessary for its functioning. The net torque is that produced by an engine that has all the accessories operating, such as fan, generator, exhaust pipe, muffler, and

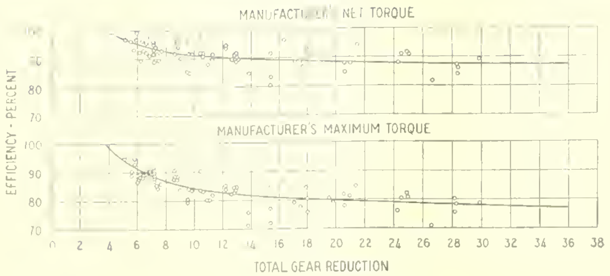

Figure 15.—Variation of Efficiency With Gear Reduction for All Trucks and Tractor-Truck Semitrailers.

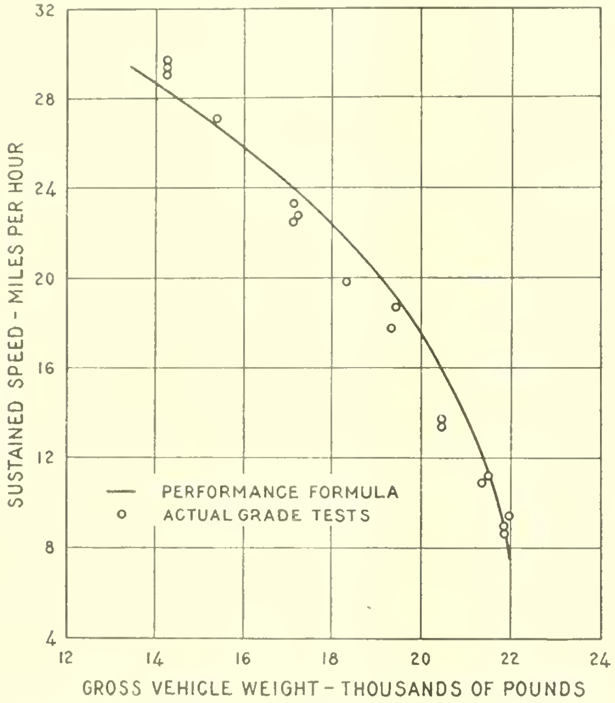

Figure 16.—Comparison of Hill-Climbing Ability as Determined by Actual Grade Tests and Performance Formula for a Tractor-Truck Semitrailer in Third Gear on a 4.5-Percent Grade.

trail pipe, that are standard or regular equipment on the engine. The engines of the chassis tested in the field were removed and tested on a cradle dynamometer to determine the net torque and horsepower available at various engine speeds at full throttle. The results of these tests were compared with the net torque and power values certified by the manufacturer and were found to be in close agreement with them in all except a few cases. In instances where there was a marked variation that could not be reasonably explained, the efficiency factors were not computed.

The determination of the maximum and net efficiency factor for one gear ratio of a vehicle is given in table 19. An average efficiency factor was obtained in this manner for each gear ratio for each of the vehicles tested. The variation of the efficiency factors with the total gear reduction for all the vehicles tested, regardless of capacity and type, is shown in figure 15 for the maximum and net torque. The values of 85 and 90 percent that are commonly used appear to be fair average values, but for particular speed conditions much more accurate values are now available. The curves drawn through the scattered points are hyperbolas, the equations of which were determined by the method of least squares. The hyperbola was found to fit the points and the conditions better than either a straight line or a parabola. The equations for the curves are:

1. For maximum torque,

$$E=74.36+\frac{99.69}{GR}.$$

2. For net torque,

$$E=85.64+\frac{56.15}{GR}.$$

The variation that occurs about the average efficiency is defined by a standard deviation of 3.5 for the maximum torque and 3.1 for the net torque. In other words, about 68 percent of the values lie within 3.5 percent of the efficiency shown by the curves.

Figure 16 compares the performance of a tractor-truck semitrailer computed by using the basic performance formula with that actually measured on a 4.5-percent grade. The net torque certified by the manufacturer and the values of efficiency and tractive resistance shown on figure 15 and table 18 were used to determine the theoretical performance. The actual performance is indicated by the plotted points. The theoretical performance is shown by the solid line. The average variation between the two sets of results is about 2.5 percent. This degree of accuracy cannot be expected in all cases, but the variation will seldom exceed 5 percent.

GRADE ABILITY OF USED TRUCKS DETERMINED BY ACCELERATION METHOD

A cheaper and shorter method for determining grade ability was desired for use in the used-truck study. Of the various methods investigated,[1] the acceleration method proved to be the most satisfactory. Its accuracy was checked by using it to determine the grade ability for each new truck for the grades on which the actual grade tests were made and by comparing the results determined by the two methods.

The drawbar pull or force produced at the driving wheels of an accelerating vehicle at a given road speed is a function of the acceleration at the given road speed and the mass of the vehicle. The following equation expresses the relationship:

$$P=a(m+k_x)\text{ - - - - - - - - - - - - - - - }(11)$$

where

P=drawbar pull in pounds,

a=linear acceleration of vehicle in feet per second per second,

m=mass of vehicle, and

k_x=mass equivalent constant for a given transmission gear.

The mass equivalent constant, which is used to compensate for the energy stored up by the rotating parts when a vehicle is accelerating or decelerating, was discussed and defined in the discussion of tractive resistance. In the case of a vehicle accelerating in a given gear the mass equivalent constant can be determined experimentally by equating the power output on the

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[1] Comparison of Methods for Determining the Hill Climbing Ability of Trucks, by C. C. Saal, PUBLIC ROADS, February 1939, p. 233.

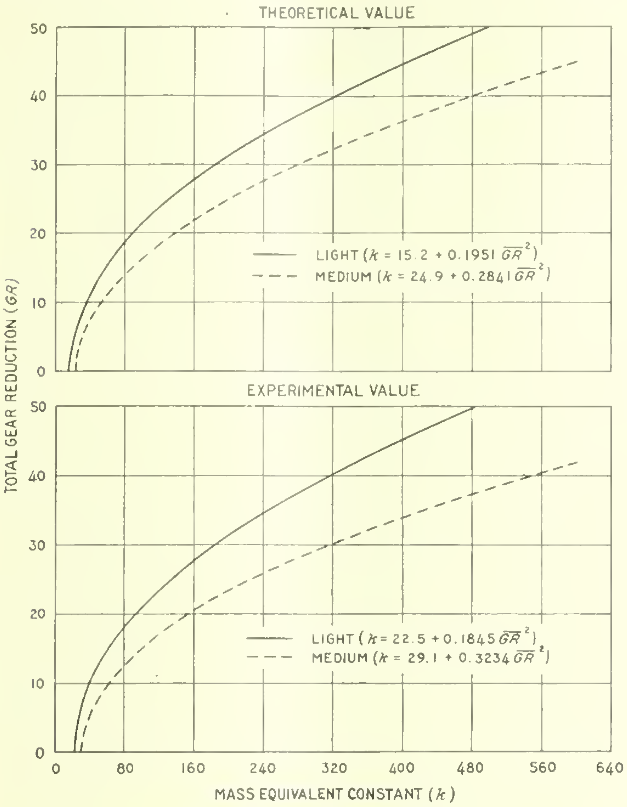

Figure 17.—Mass Equivalent Constants for Light and Medium Single-Unit Trucks.

level at a given speed and weight to the power output on a known grade at the same speed and weight. The following equation results:

$$W \sin A + a_g(m+k_x) = a_l(m+k_x)$$

or

$$k_x = \frac{W \sin A}{a_l - a_g} - m \text{ -------------- } (12)$$

where

W=weight of vehicle in pounds,

A=angle in degrees that grade line makes with horizontal,

a_g=linear acceleration on grade in feet per second per second,

m=mass of vehicle$=\frac{W}{32.2}$,

a_l=linear acceleration on level in feet per second per second, and

k_x=mass equivalent constant for a given transmission gear.

The mass equivalent constant can be determined theoretically by adding the constant determined for the vehicle in neutral gear to that for the crankshaft, flywheel, and clutch. It is determined by the following equation:

$$k_x = k_0 + \frac{I\overline{GR}^2}{r^2} \text{ -------------- } (13)$$

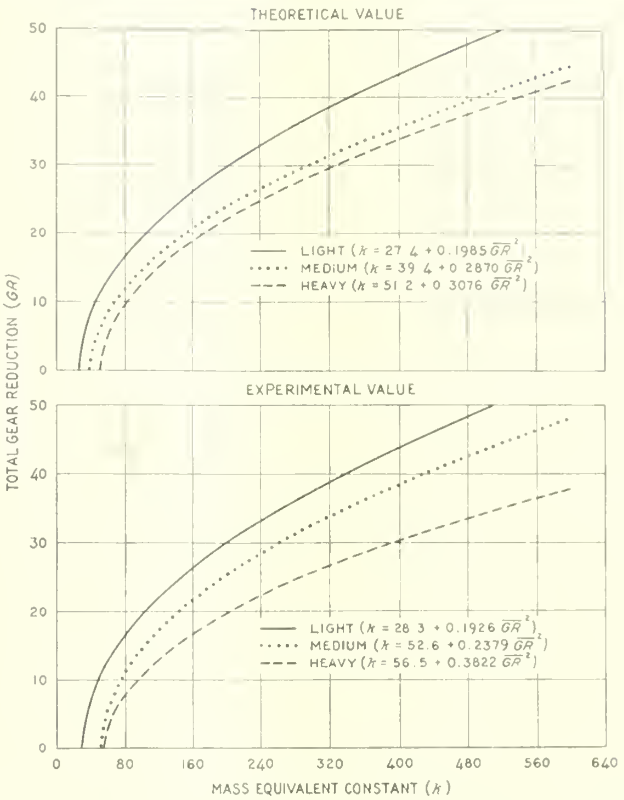

Figure 18.—Mass Equivalent Constants for Light, Medium, and Heavy Tractor-Truck Semitrailers.

where

k_x=mass equivalent constant for a given transmission gear,

k_0=mass equivalent constant for neutral gear,

I=moment of inertia of rotating parts,

GR=total gear reduction, and

r=effective radius of rotating parts in inches.

The average experimental and theoretical mass equivalent constants (k) by capacity groups and vehicle type are shown in figures 17 and 18.

The total force or tractive effort produced at the tire surface of the driving wheels at a given speed is equal to the drawbar pull plus the tractive resistance of the test vehicle. This force can be utilized to pull a certain gross vehicle weight up a given grade at the road speed for which the force is measured. In order to compute grade ability the force must be equated to the component of the gross vehicle weight parallel to the gradient and the tractive resistance of the vehicle on the grade. The following equation expresses this relation:

$$P + GVW_1f_1 = GVW_2g + GVW_2f_2$$

or

$$GVW_2 = \frac{TE}{(g+f_2)} \text{ -------------- } (14)$$

where

GVW_2=gross vehicle weight in pounds that can be carried on a given grade.

TE=tractive effort$=P+GVW_1f_1$.

$P=a\ (m+k_x)$=the drawbar pull in pounds.

GVW_1=gross weight of test vehicle in pounds.

f_1=coefficient of tractive resistance in pounds

per pound for the speed and weight for which the drawbar pull was measured,
f_2 = coefficient of tractive resistance in pounds per pound for the weight that can be carried on the grade, and
g = grade in feet of rise per foot.

There are two unknowns in the above equation, the gross vehicle weight that can be hauled up a given grade at a sustained speed and the coefficient of tractive resistance for that weight. In other words, before the coefficient (f_2) can be determined accurately, the gross vehicle weight (GVW_2) must be known. Since this is impossible, a trial computation is made assuming (f_2) to be equal to (f_1). Using the trial weight, a new coefficient is obtained with which a second computation of weight is made. The weight computed on the second trial seldom varies enough from the first one to cause a variation in the coefficient that would necessitate another trial computation.

Each test vehicle was accelerated at full throttle in each transmission gear, starting at the slowest speed at which the engine would operate smoothly and ending at the maximum permissible engine speed. The tests were made on a level section of highway and also on a 4.5-percent grade. The tests on the grade were necessary in order to determine the constant for the energy stored by the rotating parts. A time-distance record was obtained for each test run from which the values of acceleration were determined in the same manner as described for the determination of the values of deceleration.

SAMPLE CALCULATION OF GRADE ABILITY USING ACCELERATION VALUES

An illustration of the computation of the grade ability from values of acceleration is given in table 20. The values of acceleration were obtained for a tractor-truck semitrailer operating in third gear with a weight of 12,000 pounds. The effective mass is composed of the mass of the loaded tractor-truck semitrailer (373) and the mass equivalent constant (64). At 20 miles per hour the acceleration is 2.210 feet per second per second. The drawbar pull is therefore 2.210 (373+64) =966 pounds. For this particular vehicle the coefficient of tractive resistance for a weight of 12,000 pounds and a speed of 20 miles per hour is 11.4 pounds per thousand pounds. The total tractive resistance is thus (11.4×12)=137 pounds. Since the tractive effort of 1,103 pounds (966+137) is directly proportional to the power output, the correction to standard atmospheric conditions is made at this point. The factor in this case is 1.001 indicating that the tests were made at almost standard conditions. A corrected tractive effort of 1,104 pounds was used to compute the grade performance for this vehicle on 4.5- and 6-percent grades, the grades on which the actual grade tests were conducted.

From table 20 it is seen that the tractor-truck semitrailer had sufficient power to sustain a speed of 20 miles per hour on a 6-percent grade with a gross weight of 15,600 pounds. The first trial computation of the gross vehicle weight utilized the coefficient of 11.4 pounds per thousand pounds. A gross weight of 15,500 pounds resulted from the first trial. Since the coefficient for 15,500 pounds and 20 miles per hour is 10.9 instead of 11.4 pounds per thousand pounds, as assumed, a second trial was necessary. The second trial with the new coefficient resulted in a weight of 15,600 pounds, for which the coefficient is still 10.9. A third trial was therefore not necessary.

Figure 19 shows a comparison between the grade abilities determined for a tractor-truck semitrailer on a 6-percent grade. The plotted points indicate the results determined by the actual tests. The solid line shows the results determined by the acceleration tests. The variation between the two sets of results is of small magnitude. This is a fair sample of the accuracy that can be expected.

Figure 20 shows a cumulative frequency distribution of the percentage of variation between the results obtained by the two methods for all the vehicles. In almost 80 percent of the cases the variation was less than 5 percent. The acceleration method was considered accurate enough to be used in the controlled tests of used trucks and tractor-trucks.

The purpose of the tests on used trucks was to determine the effect of wear and mileage on the performance of motor trucks. Acceleration tests were made on used trucks, of the same make and model as the new trucks tested, to obtain a factor that would represent the decrease in performance resulting from wear and mileage. The Maryland Motor Truck Association cooperated to the fullest extent in this phase of the study. Their members furnished, without compensation, 17 vehicles that had mileages ranging from 10,000 to 90,000 miles. The grade abilities of these vehicles were compared with those of the new trucks to determine the effect of use on their performance.

TABLE 20.—*Grade ability by acceleration method for a tractor truck semitrailer operating in third gear*

[Correction factor=1.001. GVW=12,000 lb. M=373. K_3=64. $M+K_3$=437]

Speed (m. p. h.)	Acceleration, a	Drawbar pull 437a	Tractive resistance f_1	Actual tractive effort	Corrected tractive effort	4.5=percent grade			6-percent grade		
						GVW	Tractive resistance f_2	GVW using f_2	GVW	Tractive resistance f_3	GVW using f_3
	Ft./sec./sec.	*Pounds*	*Lb./1,000 lb.*	*Pounds*	*Pounds*	*Pounds*	*Lb./1,000 lb.*	*Pounds*	*Pounds*	*Lb./1,000 lb.*	*Pounds*
8	2.540	1,110	7.7	1,202	1,203	22,900	7.9	22,800	17,800	7.8	17,800
10	2.512	1,098	8.2	1,196	1,197	22,500	8.3	22,500	17,600	8.2	17,600
12	2.472	1,080	8.6	1,183	1,184	22,100	8.7	22,100	17,300	8.7	17,200
14	2.421	1,058	9.3	1,170	1,171	21,600	9.2	21,600	16,900	9.2	16,900
16	2.361	1,032	10.0	1,152	1,153	21,000	9.5	21,200	16,500	9.7	16,600
18	2.290	1,001	10.7	1,129	1,130	20,300	9.9	20,600	16,000	10.2	16,100
20	2.210	966	11.4	1,103	1,104	19,600	10.5	19,900	15,500	10.9	15,600
22	2.120	926	12.2	1,072	1,073	18,800	11.0	19,200	14,900	11.6	15,000
24	2.025	885	13.0	1,041	1,042	18,000	11.8	18,300	14,300	12.4	14,400
26	1.920	839	13.9	1,016	1,017	17,300	12.5	17,700	13,800	13.4	13,900
28	1.810	791	14.8	969	970	16,200	13.6	16,600	13,000	14.5	13,000
30	1.690	739	15.8	929	930	15,300	14.7	15,600	12,300	15.6	12,300
32	1.568	685	16.9	888	889	14,400	16.0	14,600	11,600	17.2	11,500

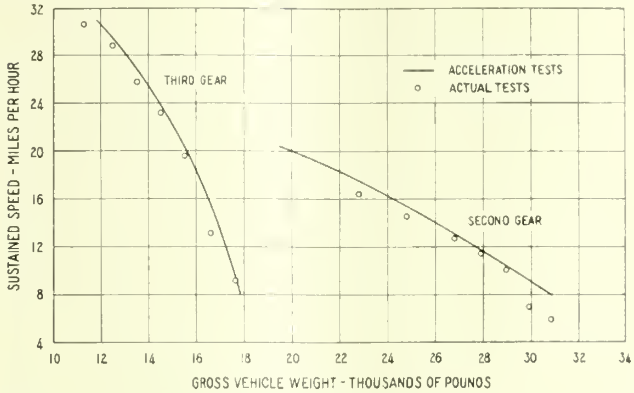

Figure 19.—Comparison of Grade Abilities Determined by Actual Grade Tests and by Acceleration Tests for a Typical Tractor-Truck Semitrailer on a 6-Percent Grade.

The acceleration method which was developed in conjunction with the actual grade tests on new trucks and which has already been described was used to measure the performance of the used vehicles. Because it was not always possible to obtain vehicles of the same model that had the same gear ratios and tire sizes, it was necessary to compare the performance of the new and used vehicles measured in terms of tractive effort at a given engine speed rather than in terms of gross vehicle weight and speed on a given grade.

The tractive effort of a new vehicle operating on a given grade and in a given gear was computed for various road speeds by equation 2:

$$TE=GVW(f+g) \qquad (2)$$

where

TE = tractive effort in pounds,
GVW = gross vehicle weight in pounds,
f = coefficient of tractive resistance in pounds per pound of weight, and
g = grade in feet of rise per foot.

The engine speed equivalent to the road speed for which the tractive effort was measured was determined by use of equation 10.

$$RPM=\frac{168\times GR\times S}{r} \qquad (10)$$

where

RPM = engine speed in revolutions per minute,
S = road speed in miles per hour,
GR = total gear reduction,
r = effective radius of driving wheels in inches, and
168 = factor to convert units to revolutions per minute.

TRACTIVE EFFORT-ENGINE SPEED CURVES PLOTTED FOR USED VEHICLES

The tractive effort of a used vehicle was measured by acceleration and deceleration tests made on a level section of road. The drawbar pull and the tractive resistance determined from the values of acceleration and deceleration were combined to obtain the tractive effort available at various engine speeds. A tractive effort–engine speed curve was then plotted for each transmission gear. The following equation was used to compute the tractive effort for the used trucks:

$$TE=a(m+k_x)+GVW(f) \qquad (15)$$

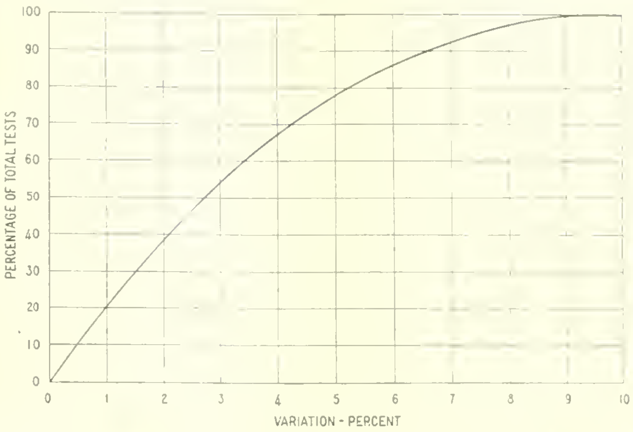

Figure 20.—Cumulative Frequency Distribution of the Percentage of Variation Between the Grade Ability Determined by the Actual Grade Tests and by the Acceleration Tests.

where

TE = tractive effort in pounds,
a = linear acceleration of vehicle in feet per second per second,
m = mass of vehicle $=\frac{GVW}{32.2}$
k_x = mass equivalent constant for a given gear,
$a(m+k_x)$ = drawbar pull in pounds,
GVW = gross vehicle weight in pounds,
f = coefficient of tractive resistance in pounds per pound of weight, and
$GVW(f)$ = tractive resistance in pounds.

In the case where the used models had the same tire size and gear ratios as the new model, the comparison of performances for a given gear is a direct one between the tractive efforts at a given engine speed or road speed, computed by using equations 2 and 15 shown above. But when the final gear reductions are not the same, it is necessary to convert the tractive effort of the used model for one gear reduction to that for the gear reduction of the new model.

The relation that was used to convert the tractive effort of a used model to one that could be compared with that of the new model was derived as follows, starting with the basic performance formula (equation 8):

$$GVW=\frac{T\times GR\times E}{r(f+g)} \qquad (8)$$

since

$$GVW(f+g)=TE \qquad (2)$$

$$TE=\frac{T\times GR\times E}{r} \qquad (16)$$

therefore for one set of conditions

$$TE_1=\frac{T\times GR_1\times E_1}{r_1} \qquad (17)$$

and for another set of conditions

$$TE_2=\frac{T\times GR_2\times E_2}{r_2} \qquad (18)$$

Dividing equation 17 by equation 18 and assuming that for similar engines the torque, T in the equations, will be the same at a given engine speed and that the

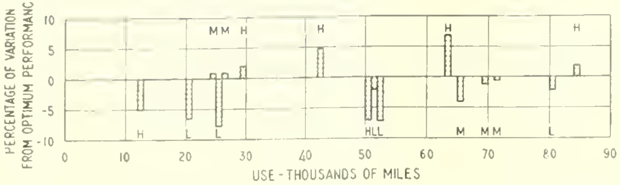

Figure 21.—The Effect of Use on the Hill-Climbing Ability of Motor Trucks.

over-all efficiency will be equal, which is the case when the variation in gear reduction is not great, the following relationship results:

$$TE_1 = TE_2\left(\frac{GR_1\times r_2}{GR_2\times r_1}\right) \qquad (19)$$

where

TE_1 = converted tractive effort of used model.
TE_2 = tractive effort determined for used model.
GR_1 = total gear reduction of new model.
GR_2 = total gear reduction of used model.
r_1 = loaded radius of driving wheels on new model in inches, and
r_2 = loaded radius of driving wheels on used model in inches.

For example, one new model produced a tractive effort of 1,522 pounds at 2,200 revolutions per minute with a final gear reduction of 12.85 and a loaded radius of 19.11 inches. The used model with the same engine produced a tractive effort of 1,442 pounds at 2,200 revolutions per minute with a final gear reduction of 12.89 and a loaded radius of 19.85 inches. Using equation 19:

$$TE_1 = 1{,}442\left(\frac{12.85\times 19.85}{12.89\times 19.11}\right) = 1{,}493 \text{ pounds.}$$

The variation between the performance of the new and used model is therefore $\frac{1{,}522-1{,}493}{1{,}522}=0.019$, or a reduction in performance due to use of 1.9 percent, since the tractive efforts of both chassis are for the same set of conditions of engine speed, gear reduction, and loaded tire radius.

The percentage of variation between the tractive effort produced by the new and used models was computed at several engine speeds over the useful speed range of at least one gear. The procedure followed and the results of the tests on a new truck and two used trucks of the same make and model are shown in table 21.

The performance of the new truck was obtained from the results of the actual grade tests on a 5-percent grade. The tractive effort was computed by multiplying the gross vehicle weight by the factor $(f+g)$ which is the sum of the coefficient of tractive resistance and the tangent of the angle of grade.

The tractive effort determined for the used truck by the acceleration method is plotted against engine speed. From these curves the tractive effort shown in the table was obtained for the engine speed at which the tractive effort for the new truck was determined. This tractive effort, designated as TE_2 in the table, was converted to the gear reduction and loaded tire radius of the new truck in the manner just described. The percentage of variation between the performance of the new and used vehicle was then computed using the performance of the new truck as a base.

Table 21.—*Comparison of the tractive effort for new and used trucks of the same make and model*

	Performance of new truck on a 5-percent grade					Performance of used truck with 12,000 miles				Performance of used truck with 51,000 miles			
Speed, m. p. h.	GVW	Speed	f	$f+g$	TE	TE_2	TE_1	Variation		TE_2	TE_1	Variation	
	1,000 Lb.	*R. p. m.*	*Lb./1,000 lb.*		*Lb.*	*Lb.*	*Lb.*	*Lb.*	*Percent*	*Lb.*	*Lb.*	*Lb.*	*Percent*
10	30.6	1,060	9.9	0.0599	1,835	1,770	1,755	80	−4.4	2,090	1,745	90	−4.9
12	30.1	1,275	10.4	.0604	1,820	1,750	1,735	85	−4.7	2,050	1,710	110	−6.0
14	29.4	1,485	10.9	.0609	1,790	1,720	1,705	85	−4.7	2,010	1,680	110	−6.1
16	28.4	1,690	11.6	.0616	1,750	1,680	1,660	90	−5.1	1,950	1,630	120	−6.8
18	27.2	1,900	12.3	.0623	1,695	1,620	1,600	95	−5.6	1,880	1,570	125	−7.4
20	25.6	2,110	13.3	.0633	1,620	1,550	1,530	90	−5.6	1,790	1,495	125	−7.7
22	23.8	2,320	14.3	.0643	1,530	1,470	1,450	80	−5.2	1,680	1,405	125	−8.2
24	22.0	2,520	15.5	.0655	1,440	1,380	1,355	85	−5.9	1,570	1,310	130	−9.0
26	20.0	2,730	17.0	.0670	1,340	1,280	1,255	85	−6.3	1,440	1,200	140	−10.4
28	17.5	2,940	18.8	.0688	1,205	1,180	1,160	45	−3.7	1,310	1,095	110	−9.1
Average variation									−5.1				−7.6

TRUCK PERFORMANCE NEED NOT DECREASE WITH USE

The percentage of variation between the performance of new and used trucks is shown graphically in figure 21 for all the tests. The abscissa is the mileage the vehicle had been driven and the ordinate is the percentage of variation from the base performance. The variation from the base line is never more than 8 percent. However, it is not always in the negative direction. In about one-third of the cases the performance of the used truck was better than that of the new truck. The bars are marked L, M, and H, to indicate that the test vehicle was in the light, medium, or heavy group. It is interesting to note that the heavier models of used vehicles had performances better than those of new vehicles. Since a majority of the vehicles were obtained from operators who service their motors regularly, the performance of the used vehicles may have been of a higher standard than that found generally on the highway. The results definitely prove that trucks can be maintained so that the performance does not decrease with reasonable use.

The amount of maintenance on the used trucks should be known when considering the effect of use on the performance. Some of the motors of vehicles tested had been overhauled prior to the tests. The medium truck with 69,000 miles was given a major overhaul at 54,700 miles, and the medium truck with 65,000 miles was overhauled at 23,000 miles because sand had gotten into the motor. Both of these vehicles were used to haul material from a sand pit and their performance as shown on figure 21 is slightly less than the base performance. The light vehicle with 80,000 miles was overhauled at about 40,000 miles. Its performance is also about the same as that of a new vehicle. In one instance a light vehicle was tested immediately before and again immediately after overhauling. The performance after the overhaul was slightly less than that before the overhaul as is indicated by the values of variation plotted at 51,000 and 52,000 miles. None of the other vehicles were overhauled prior to the tests.

In two cases the used models had the same gear reduction and tire size as the new model. In these cases a direct comparison could be made between the grade performances in terms of gross vehicle weight and speed. Figures 22 and 23 show the comparison of grade ability

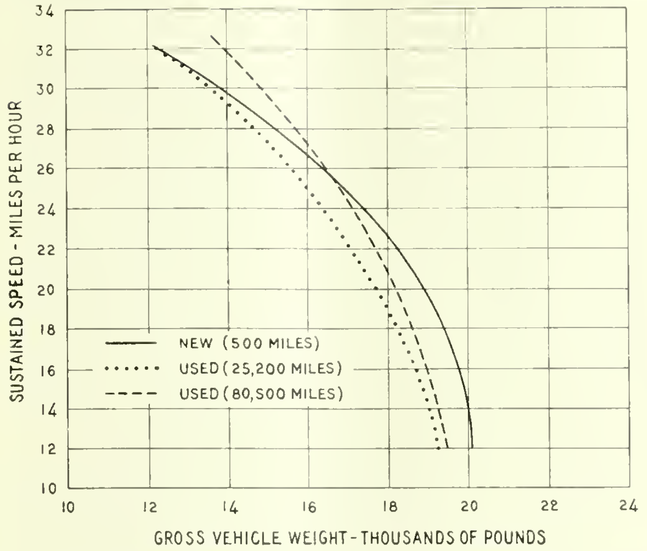

Figure 22.—Comparison of the Hill-Climbing Ability in Third Gear of New and Used Light Tractor-Truck Semitrailers of the Same Make and Model on a 5-Percent Grade.

for groups of light and medium tractor trucks. These comparisons serve to verify the conclusions drawn from the results shown in figure 21.

In figure 22 the performance curve of the used vehicle with 80,500 miles crosses that for the new vehicle and is better than the performance of the vehicle with 25,200 miles. The effect of use is practically nil in this case.

In figure 23 the performance curves of the used vehicles cross the one for the new vehicle. The average variation between the curves for the two used vehicles and the one for the new vehicle is approximately zero when negative variation is weighed against positive variation.

The purpose of the tests on vehicles in service was to measure the behavior of other vehicles in the traffic stream with respect to the slow-moving vehicles and the hill-climbing ability of motor trucks when operated under ordinary driving. These tests have been conducted on various gradients under different traffic densities in Massachusetts, Illinois, California and Oregon, in connection with their respective highway planning surveys. Included in this report are the results found in Massachusetts and California. The analysis of data in the vehicle behavior portion of the study, which has as its principal objective the determination of what may be considered a reasonable minimum speed from the point of view of congestion, has not been concluded.

The actual performance of all vehicles on the selected grades was measured with the equipment used on passing studies.[2] The trucks and combination units were identified when they passed a control station on the grade. After they had climbed the grade they were stopped at a loadometer station where they were weighed and complete data were obtained, as to make, year, model, capacity, vehicle type, tire size, etc. In this way it was possible to match the speed record of the vehicle with the vehicle characteristics without permitting the operator to know that his actions were being observed on the grade.

In analyzing the charts each vehicle was treated in the order in which it arrived at the grade. The average speed of each truck was measured for each 50-foot section. From this record of speed it was possible to obtain the sustained or crawl speed and also the length of grade required to reduce the approach speed to various speeds.

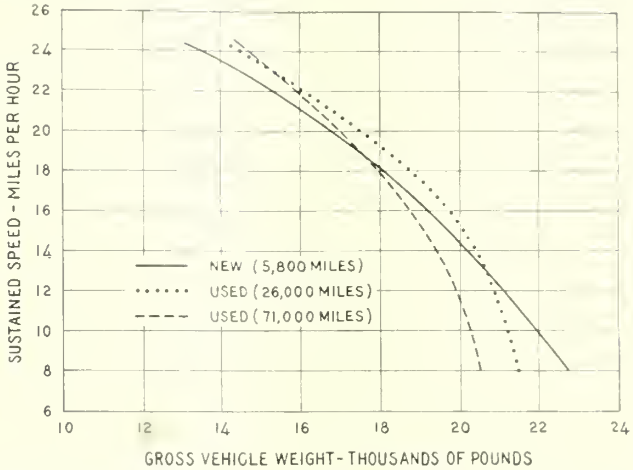

Figure 23.—Comparison of the Hill-Climbing Ability in Third Gear of New and Used Medium Tractor-Truck Semitrailers of the Same Make and Model on a 6-Percent Grade.

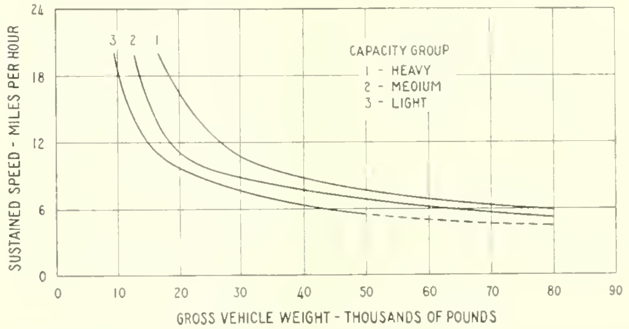

Figure 24.—Grade Ability of Light, Medium, and Heavy Trucks and Tractor-Truck Semitrailers in Actual Service on a 6-Percent Grade.

The trucks and combination units observed were classified into the three capacity groups corresponding to those used in the new truck study. The vehicles in each capacity group were distributed into weight groups and the average sustained speed obtained for each weight group. Figure 24 shows the average performance of 517 vehicles observed in Massachusetts and California on 6-percent grades. Above 30,000 pounds there is little difference in the speed for a given weight between the performance of vehicles in the three capacity groups. This finding is particularly important when it is considered that an increase in power is one of the means that has been considered for raising the speed.

From the performance curves for the new trucks, a requirement of 20 miles per hour on a 4-percent grade, one that has been proposed in some quarters, is indicated to be equivalent to one of 14 miles per hour on a 6-percent grade. From figure 24, this speed could be maintained with gross vehicle weights of 12,500, 16,000 and 22,500 pounds for the light, medium, and heavy vehicles, respectively. These weights are approximately the empty weights of the combination units. The effect of such a requirement based on actual operation on the tractor-truck semitrailers is apparent. It would permit no carried load whatsoever.

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[2] Procedure Employed in Analyzing Passing Practices of Motor Vehicles, by F. H. Holmes, PUBLIC ROADS, vol. 19, No. 11, January 1939.

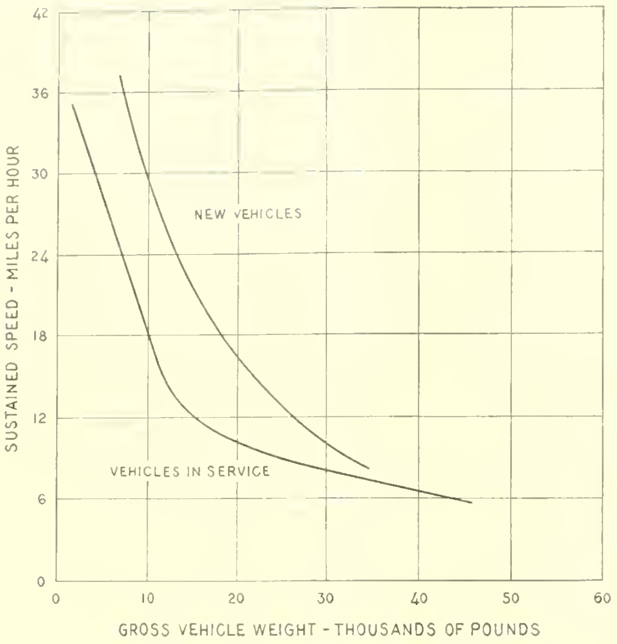


Figure 25.—Comparison Between the Grade Ability Determined by Controlled Tests on New Vehicles and by Tests on Vehicles in Actual Service for Light Trucks and Tractor-Truck Semitrailers on a 6-Percent Grade.

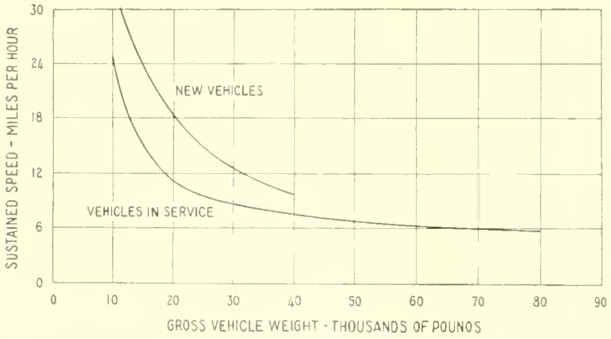


Figure 26.—Comparison Between the Grade Ability Determined by Controlled Tests on New Vehicles and by Tests on Vehicles in Actual Service for Medium Trucks and Tractor-Truck Semitrailers on a 6-Percent Grade.

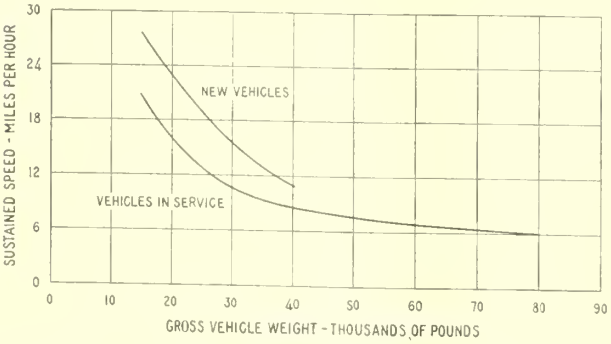


Figure 27.—Comparison Between the Grade Ability Determined by Controlled Tests on New Vehicles and by Tests on Vehicles in Actual Service for Heavy Trucks and Tractor-Truck Semitrailers on a 6-Percent Grade.

INFERIOR PERFORMANCE CAUSED BY IMPROPER DRIVING

A direct comparison between the performance of new trucks and that of the vehicles in service on a 6-percent grade is shown in figures 25, 26, and 27, for the light, medium, and heavy trucks and combination units. The difference in performance is caused either by improper maintenance or improper driving or both. In each figure the two curves tend to close at the lower speeds, which indicates that much of the difference is due to the improper selection of gears, a fact that is further supported by the results of the used-truck study. The average variations are 31, 30, and 27 percent for the light, medium, and heavy groups, respectively. An average variation of 30 percent can therefore be expected for all the vehicles studied. This decrease in performance must be given careful consideration in any development of a reasonable performance requirement.

The effect of the length of grade on the speed of motor trucks is given by figure 28 for a 6-percent grade studied in Massachusetts. The curve for each gross weight group was obtained by averaging the speeds of all the vehicles in a group at a given distance along the grade. These curves can be used by the designer to determine how long a grade can be and yet not reduce the speed below a given value. For instance, a 6-percent grade could be 800 feet long and yet not slow the average vehicle to less than 20 miles per hour. The results apply to a grade with a level approach. Similar results are being developed for other gradients having different rates of grade and approach characteristics.

The correlation of the results of the actual grade tests with the information obtained at weight stations to determine the effect of various requirements on the present operation of motor trucks is one of the important developments of this study. The number of vehicles affected and the tons of payload that would be lost to the operators using U. S. Routes 1 and 40 between Richmond, Virginia, and Havre de Grace, Maryland have been determined for certain assumed requirements. The weight of the vehicles, their capacity (light, medium, or heavy) and their type were recorded at weight stations. The maximum performance that can be expected from these vehicles is shown in table 22 for various conditions of speed and grade as determined by the actual grade tests on new vehicles. The number of vehicles that cannot meet a specified requirement, such as 20 miles per hour on a 4-percent grade, was determined by

Table 22.—*Grade ability of light, medium, and heavy trucks and tractor-truck semitrailers on various grades*

Grade, percent	Capacity	Gross vehicle weights for speeds of—		
		15 miles per hour	20 miles per hour	25 miles per hour
		Pounds	*Pounds*	*Pounds*
3	Light	39,000	26,000	22,000
	Medium	42,000	33,000	23,000
	Heavy	54,000	41,000	31,000
4	Light	31,000	21,000	18,000
	Medium	34,000	26,000	19,000
	Heavy	44,000	33,000	25,000
5	Light	26,000	17,000	15,000
	Medium	29,000	22,000	16,000
	Heavy	36,000	27,000	20,000
6	Light	22,000	15,000	13,000
	Medium	24,000	19,000	14,000
	Heavy	31,000	23,000	17,000

counting the vehicles that had weights that were more than the weight given in table 22 for a given set of conditions.

The percentage of the single-unit trucks and tractor-truck semitrailers that must decrease their payload to meet a requirement of 20 miles per hour on a 4-percent grade is shown by the bar graphs in figure 29. The variation in performance between the two types of vehicles is very pronounced. Approximately 65 percent of the combination units would be affected as compared to about 5 percent of the single units. Similar relationships have been determined for other conditions of speed and grade. Table 23 lists the percentage of light, medium, and heavy vehicles using U. S. Route 1 between Washington, D. C. and Baltimore, Md., that cannot meet various requirements of speed and grade. For a requirement of 20 miles per hour on a 3-percent grade, 27.2 percent of the combination units would be affected as compared to 0.8 percent of the single units. For 20 miles per hour on a 5-percent grade, 80.9 percent of the combination units would be affected as compared to 10.6 percent of the single units. It is evident that the operators of the tractor-truck semitrailers would suffer the most.

The figure and the table just reviewed indicate the number of vehicles that are affected, but they do not show in what measure they are affected. The amount of payload that would have to be removed from the vehicles to permit them to meet a given requirement is more important. Table 24 lists the total payload hauled by all the vehicles operating between Washington, D. C., and Baltimore, Md., for an average 24-hour period. It also lists the payload in excess of that which could be carried if a speed of 20 miles per hour were required on 3-, 4-, 5-, or 6-percent grades.

TABLE 23.—*Percentage of light, medium, and heavy vehicles between Washington, D. C., and Baltimore, Md., that cannot meet various requirements of speed and grade.*

Requirement		Percentage of single-unit trucks that cannot meet requirement				Percentage of tractor-truck semitrailers that cannot meet requirement			
Speed, m. p. h.	Grade	Light	Medium	Heavy	Average	Light	Medium	Heavy	Average
	Percent								
15	3	0	0	0	0	0	2.8	0	1.5
	4	0	.7	0	.2	6.6	22.0	0	14.0
	5	.6	1.9	8.4	1.3	34.2	51.3	25.6	42.2
	6	2.1	7.6	13.3	4.2	60.9	72.4	59.0	66.8
20	3	.6	1.7	0	.8	34.2	28.4	0	27.2
	4	2.8	3.6	13.3	3.5	68.3	64.3	45.0	63.5
	5	8.5	14.0	21.7	10.6	83.1	79.1	82.1	80.9
	6	17.8	26.9	39.8	21.2	88.5	88.9	94.9	89.4
25	3	2.1	10.9	13.3	5.0	60.9	76.0	59.0	68.7
	4	6.4	26.9	31.3	13.0	80.2	88.9	91.0	86.0
	5	17.8	48.6	66.3	28.5	88.5	96.1	98.7	93.7
	6	32.1	60.9	84.3	42.5	93.4	97.8	100.0	96.5

For this requirement on a 4-percent grade the total payload must be reduced 14.4 percent with 13.5 percent of this coming off the tractor-truck semitrailers. For a 3-percent grade, the payload must be reduced 3.4 percent. For the 5- and 6-percent grades, 28.0 and 39.8 percent of the payload must be removed. In either case at least 85 percent of the excess load is found on the combination units. The translation of the excess load into terms of ton-miles and lost income can easily be made. The effect of these requirements on the transportation industry, even if they were sufficient to eliminate the congestion on grades, would be drastic. In

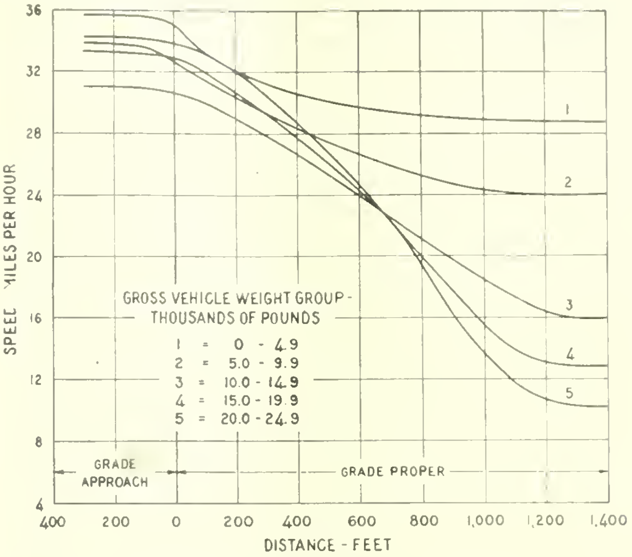


FIGURE 28.—EFFECT OF THE LENGTH OF A 6-PERCENT GRADE ON THE SPEED OF MOTOR TRUCKS.

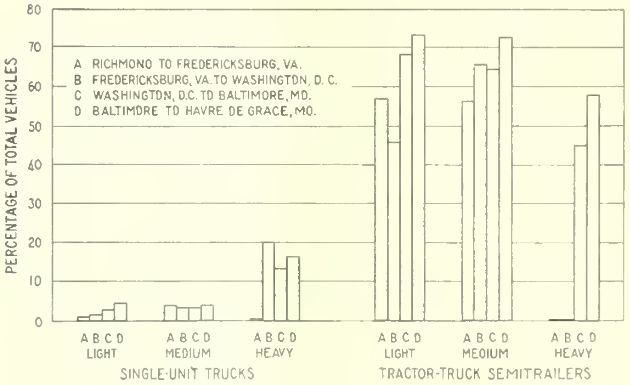


FIGURE 29.—PERCENTAGE OF TOTAL VEHICLES THAT CANNOT MEET REQUIREMENT OF 20 MILES PER HOUR ON A 4-PERCENT GRADE.

TABLE 24.—*Average excess load for trucks and tractor-truck semitrailer combinations on 3-, 4-, 5-, and 6-percent grades at 20 miles per hour; 24-hour daily average traffic on U. S. Route 1 between Washington, D. C., and Baltimore, Md.*

Type and capacity of vehicle	24-hour volume	Total pay load	Excess load for various grades							
			3 percent		4 percent		5 percent		6 percent	
		Tons	*Tons*	*Percent*[1]	*Tons*	*Percent*[1]	*Tons*	*Percent*[1]	*Tons*	*Percent*[1]
Light trucks	1,084	2,380	2	0.02	31	0.3	122	1.4	233	2.6
Medium trucks	422	1,220	7	.08	34	.4	94	1.1	202	2.3
Heavy trucks	83	438	-----	-----	15	.2	51	.5	100	1.1
Total trucks	1,589	4,038	9	.10	80	.9	267	3.0	535	6.
Light tractor-truck semitrailers	243	1,530	114	1.3	400	4.5	770	8.6	965	10.9
Medium tractor-truck semitrailers	359	2,715	178	2.0	750	8.4	1,255	14.2	1,700	19.2
Heavy tractor-truck semitrailers	78	565	-----	-----	52	.6	197	2.2	334	3.7
Total tractor semitrailers	680	4,810	292	3.3	1,202	13.5	2,222	25.0	2,999	33.8
Total	2,269	8,848	301	3.4	1,282	14.4	2,489	28.0	3,534	39.8

[1] Percentage of total payload hauled by all vehicles.

studying this table it is important to note that, while the combination units represent only 30 percent of the total number of vehicles, they haul 55 percent of the total payload.

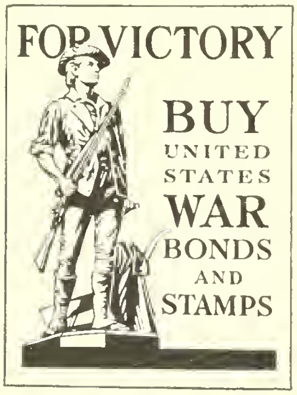
CONCLUSIONS

The most important conclusions are derived from the results of the actual grade tests and the applications just described. They showed that for motor trucks even to approach reasonable speeds on grades:

1. Grades must be reduced to 3 percent or less, or
2. Engine power must be more than doubled, or
3. Gross vehicle weights must be reduced excessively, or
4. Some combination of the three must be used that will still be costly to all interests involved and practically impossible of immediate application. Before a final conclusion can be reached, the reasonable minimum speed must be determined and the relative economics of the three basic methods and of their combinations must be determined. The results do show plainly that it will not be possible to find a comprehensive solution. It appears that the immediate solution is the localized one—the provision of wider surfaces at the points of most serious congestion. This does not mean, however, that emphasis should be removed from the other methods as a means of gradually improving the performance of motor trucks.

On the average, there is a 30-percent variation between the possible performance and the actual performance of vehicles in service. Since not over 10 percent of this variation should be due to lack of maintenance, there remains a 20-percent variation that must be charged to improper operation of the vehicle. In some cases the speed could have been doubled if the gears had been shifted at the proper engine speed. A considerable improvement in performance could be realized merely by instructing the drivers as to the proper speeds at which to shift gears.

The performance of a motor vehicle may be computed by using the unit tractive resistances and efficiencies developed by this study, with a degree of accuracy not heretofore possible. Further research is necessary to determine the tractive resistance for higher speeds, for motor trucks with other types of bodies, and for various types of road surface.



STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF APRIL 30, 1942

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$6,456,126	$3,256,172	228.2	$4,278,809	$2,323,129	115.2	$198,900	$99,450	0.2	$2,493,120
Arizona	1,683,959	1,241,721	68.1	1,283,758	1,048,608	47.3	238,615	188,508	1.7	1,821,467
Arkansas	3,640,340	1,662,291	57.4	1,173,174	585,470	60.1	192,792	118,191	1.7	1,706,344
California	8,996,533	4,864,759	150.1	4,900,628	3,268,188	55.5	873,300	596,634	20.7	4,187,244
Colorado	2,377,987	1,327,638	155.2	3,579,256	2,302,210	155.1	1,165,565	652,450	41.2	2,095,717
Connecticut	1,419,554	696,181	17.1	1,988,467	957,191	23.1	208,175	104,067	4.7	1,228,870
Delaware	595,356	333,581	17.1	441,010	216,559	9.0	268,040	134,020	8.4	1,462,962
Florida	1,342,189	698,897	66.7	3,130,037	1,821,323	49.2	512,242	319,028	4.6	2,915,875
Georgia	3,007,820	1,488,938	111.7	7,491,049	3,796,161	286.3	3,219,349	1,699,825	122.1	6,732,293
Idaho	1,916,219	1,264,905	95.4	1,176,165	831,418	61.4	58,224	36,000	.1	1,988,253
Illinois	4,119,357	2,001,308	89.5	7,144,698	4,170,552	121.7	1,361,200	680,600	6.2	6,514,422
Indiana	4,624,009	2,129,006	75.1	7,377,291	3,609,880	107.1				2,895,234
Iowa	3,909,378	1,887,029	170.5	4,479,059	1,800,695	136.5	407,151	86,450	6.8	2,496,741
Kansas	4,723,956	2,399,067	263.2	6,187,277	3,230,881	258.3	893,346	416,787	43.0	4,799,194
Kentucky	4,071,654	2,006,401	146.9	6,142,754	2,928,407	118.8	2,546,540	1,341,834	27.3	1,616,384
Louisiana	1,505,665	789,834	33.4	1,382,924	914,401	28.5	3,008,481	1,564,690	60.4	3,963,622
Maine	944,081	470,251	26.8	2,064,492	1,055,796	27.7	78,610	39,305	.1	1,130,475
Maryland	2,765,267	1,381,529	29.6	3,496,909	1,483,654	15.0	310,000	202,500	3.2	1,384,707
Massachusetts	2,351,882	1,178,572	17.3	2,239,503	1,159,262	14.9	1,273,400	658,873	9.1	3,817,721
Michigan	8,514,375	4,188,577	174.2	2,967,748	1,935,599	51.6	1,325,800	828,550	11.5	3,011,267
Minnesota	4,681,764	2,286,967	384.8	9,551,361	4,732,108	391.6	113,720	56,860	10.0	3,522,288
Mississippi	5,332,567	2,602,918	283.3	3,750,524	1,882,462	218.1	21,400	13,700		2,276,878
Missouri	5,218,752	2,659,656	165.0	10,654,415	6,097,445	192.4	1,930,351	665,452	25.4	3,957,120
Montana	2,170,862	1,258,436	117.1	4,203,667	2,521,973	202.5	210,734	119,823	26.6	4,403,366
Nebraska	3,062,166	1,513,744	297.7	5,092,749	2,561,329	456.7	174,888	109,530	14.2	4,414,959
Nevada	2,217,826	1,893,244	110.6	883,384	770,620	26.0	817,668	737,447	24.6	812,356
New Hampshire	343,779	211,723	6.0	1,800,559	1,022,873	20.2				818,906
New Jersey	2,998,818	1,460,264	26.6	2,988,902	1,494,371	16.2				3,051,981
New Mexico	1,874,157	1,221,453	125.0	905,551	647,891	51.1				2,753,730
New York	9,538,881	4,786,928	126.7	8,683,891	5,561,808	86.7	429,000	193,500	6.4	5,148,137
North Carolina	4,638,838	2,495,290	187.3	2,282,864	1,213,544	99.0	561,756	283,902	8.8	3,667,794
North Dakota	3,225,343	1,875,878	287.6	2,588,295	1,469,013	203.9	2,396,350	1,257,134	205.5	4,018,917
Ohio	9,983,010	5,462,688	97.9	14,619,362	7,609,744	83.8	1,970,140	870,818	19.5	2,923,526
Oklahoma	2,643,551	1,383,859	122.3	2,692,322	1,500,534	64.9	1,360,930	734,857	38.3	6,154,092
Oregon	2,783,897	1,644,626	73.0	3,200,754	1,659,318	71.4	1,061,993	802,010	37.6	1,110,688
Pennsylvania	10,764,735	5,315,777	106.2	9,735,865	5,037,114	72.1	1,300,656	831,617	9.7	4,413,736
Rhode Island	1,194,495	678,165	10.0	1,046,898	669,630	5.0	242,264	121,132	1.8	717,226
South Carolina	2,401,104	1,210,729	101.0	3,982,258	2,526,202	92.0	524,690	281,300	11.0	1,437,385
South Dakota	2,380,869	1,398,927	287.9	4,709,133	3,116,405	533.3	545,110	325,580	59.4	2,883,332
Tennessee	4,543,199	2,540,192	109.9	4,562,862	2,627,570	99.7	200,928	118,090	3.5	3,505,818
Texas	12,767,078	6,254,887	617.9	11,080,522	5,295,251	353.1	1,100,810	361,565	29.5	9,405,618
Utah	1,252,200	941,940	53.9	1,838,587	1,393,120	46.7				1,254,463
Vermont	803,276	449,292	28.8	1,193,326	715,241	20.7	36,906	18,453	.3	393,034
Virginia	4,144,660	1,921,249	74.9	3,402,388	1,843,051	57.2	35,490	17,745	.4	2,734,942
Washington	1,808,560	987,042	26.5	2,514,659	1,362,843	30.5	43,686	23,400	1.0	2,164,196
West Virginia	2,908,443	1,442,392	53.1	2,621,338	1,387,129	29.0	972,390	544,930	5.7	1,603,136
Wisconsin	2,202,064	1,146,706	92.7	5,618,496	3,567,693	163.9	655,983	482,100	11.8	3,767,879
Wyoming	1,439,453	1,027,059	148.4	1,773,023	1,317,527	123.5				1,272,971
District of Columbia	1,056,968	542,738	3.5	198,302	127,960	.9	945,456	564,688	1.5	144,168
Hawaii	141,835	75,057	2.8	1,098,684	813,357	10.9	499,587	480,115	5.2	1,424,204
Puerto Rico	499,945	238,835	3.9	2,295,515	1,173,260	18.0	258,665	138,464	3.5	606,323
TOTALS	179,988,802	94,195,318	6,125.8	204,495,464	113,157,770	5,583.3	36,551,281	19,921,974	934.2	145,025,081

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF APRIL 30, 1942

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$1,392,972	$695,108	62.5	$685,882	$362,410	31.1	$40,801	$19,550	0.1	$581,490
Arizona	255,110	184,247	14.1	138,065	101,042	6.8				513,118
Arkansas	629,977	237,235	33.1	483,854	241,861	26.5	70,062	34,701	2.4	301,121
California	1,041,542	641,734	18.3	743,259	567,493	7.6				1,031,859
Colorado	209,672	112,570	20.9	131,800	73,914	3.2	152,386	35,323	5.0	502,702
Connecticut	298,035	136,331	6.1	266,247	115,937	4.8				199,604
Delaware	164,535	79,371	.4	141,464	70,732	11.9	102,873	37,618	3.9	246,037
Florida	1,042,471	519,285	11.6	292,018	153,304	6.1				398,953
Georgia	545,666	257,833	43.4	1,205,213	694,956	74.9	400,257	200,129	43.7	1,126,812
Idaho	293,347	172,516	26.2	225,220	157,021	7.7	38,149	23,587	2.8	285,992
Illinois	1,087,608	515,800	59.4	1,168,384	584,192	54.5	96,500	48,250	10.6	896,158
Indiana	586,368	285,054	39.8	1,279,755	606,721	51.0	88,800	44,400	3.2	922,073
Iowa	640,297	295,716	165.0	560,916	253,045	73.4	141,325	64,325	35.1	589,961
Kansas	846,435	427,455	111.7	1,806,645	905,910	92.4	238,353	119,176	29.7	1,035,351
Kentucky	1,157,864	321,329	83.2	1,249,010	417,668	37.2	74,213	24,400	5.6	324,331
Louisiana	539,462	265,678	20.6	7,700	3,850		289,362	138,761	21.5	666,650
Maine	78,522	39,261	3.6	235,218	117,609	10.6	16,850	2,714	.4	160,925
Maryland	579,966	289,808	20.7	226,410	113,205	1.8				345,564
Massachusetts	179,789	93,569	4.1	688,233	377,683	10.1				547,304
Michigan	1,290,556	636,337	79.4	710,798	355,399	21.7	307,870	153,935	11.2	644,804
Minnesota	1,597,866	790,632	217.1	1,029,197	520,337	91.8	279,496	139,348	26.8	588,655
Mississippi	829,715	414,858	46.5	1,494,546	709,614	59.0				386,494
Missouri	435,434	214,924	52.9	932,442	448,633	94.0	186,244	63,193	32.4	1,045,552
Montana	375,564	211,817	58.5	292,724	170,722	31.0	13,569	7,715	4.5	901,005
Nebraska	361,584	178,543	49.4	493,954	249,591	66.6				699,809
Nevada	331,931	266,129	28.6	92,429	60,188	4.6				234,872
New Hampshire	156,054	95,143	4.7	239,437	118,819	3.6				133,643
New Jersey	487,496	239,245	9.6	464,115	256,160	13.9				575,787
New Mexico	566,935	357,245	47.7	196,149	126,792	15.2	66,176	42,419	4.4	250,178
New York	956,940	468,240	28.8	1,160,340	620,808	14.9	245,600	122,800	.9	1,085,178
North Carolina	330,711	170,040	34.9	522,407	278,748	36.3	69,820	20,000	5.0	695,703
North Dakota	29,802	15,664	2.4	7,382	7,382		808,050	793,860	42.7	755,172
Ohio	1,782,884	888,281	57.6	901,610	498,775	14.5	76,000	38,000	2.0	1,375,489
Oklahoma	400,166	210,874	27.6	27,872	14,715	4.0	1,260,706	665,688	75.9	894,506
Oregon	459,181	244,030	41.8	487,568	284,564	28.5	30,482	18,000	1.3	322,918
Pennsylvania	2,015,130	996,906	37.6	513,989	256,587	10.4	73,588	36,794	1.8	771,950
Rhode Island	227,579	118,127	2.6	15,494	11,497					130,911
South Carolina	787,356	307,866	54.6	221,700	79,945	.5				397,720
South Dakota	37,125	22,152	15.2	4,156	4,156		1,143,430	1,047,600	114.5	762,018
Tennessee	468,207	232,400	14.1	1,357,954	688,977	46.0	158,042	79,021	4.6	750,269
Texas	1,534,043	749,810	158.3	527,559	257,547	41.7				2,249,798
Utah	204,014	136,160	20.2	136,491	88,790	3.5				337,043
Vermont	40,708	18,109	1.2	222,515	117,328	8.9				53,580
Virginia	403,187	189,037	16.2	348,516	156,180	4.6	23,400	11,700	3.8	679,170
Washington	391,635	228,446	23.8	349,818	147,362	7.8				409,402
West Virginia	412,849	207,432	19.8	301,364	150,069	6.4				510,286
Wisconsin	946,341	475,307	42.7	1,411,616	644,981	46.0	88,199	38,880	1.1	582,353
Wyoming	365,617	160,649	18.8	508,423	218,112	34.3				218,048
District of Columbia	79,178	39,035	.9							163,600
Hawaii	1,158	1,096		1,279	1,279					334,875
Puerto Rico	170,027	77,541	7.5	73,642	38,810	3.1				217,968
TOTALS	30,046,611	14,931,975	1,965.7	26,582,779	13,501,420	1,224.4	6,580,603	4,071,887	496.9	29,834,761

U. S. GOVERNMENT PRINTING OFFICE: 1942

PUBLICATIONS of the PUBLIC ROADS ADMINISTRATION

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Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

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STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF APRIL 30, 1942

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$151,770	$151,234	2	4	2	$390,025	$386,803	5	2	2	$181,625	$181,235	4		5	$997,199
Arizona	184,378	184,361	1	1		138,529	129,838		1	2	4,095	4,095			1	230,684
Arkansas	474,307	471,932	5		9	169,826	168,116	1		2	7,394	7,394			2	687,407
California	838,839	649,267	2	1	6	870,516	868,399	8			7,529	7,529			2	2,319,499
Colorado	5,685	5,646				670,460	670,460	7		3	14,914	14,914			7	745,747
Connecticut	166,222	165,415	2			61,712	60,676		1		231,374	222,740		1		540,683
Delaware	89,725	89,725	1		1	191,599	189,867	1	1		508,406	321,785	2			179,564
Florida	120,961	120,961			20	843,067	840,887	8		6	82,492	82,432			14	871,382
Georgia	668,629	668,629	7	2	6	1,002,916	1,002,916	6	6	13	790,188	790,188	2	4	10	1,505,777
Idaho	27,850	23,603			3	322,273	313,602	4	1		4,190	4,190			2	427,671
Illinois	851,050	598,683	2		95	1,648,238	1,565,041	8	1	2	385,449	385,249	1	1	31	2,546,395
Indiana	611,831	611,831	6		28	474,802	469,093	2	1	17	76,703	76,703			23	1,165,932
Iowa	375,728	365,422	3	1	33	1,432,878	1,180,856	10	2	11	212,237	204,790	1		44	621,957
Kansas	89,963	80,557	2		7	745,377	745,377	9		4	141,011	141,011	3		6	1,304,511
Kentucky	1,109,873	1,107,509	9		1	441,711	441,711	4	1							411,382
Louisiana	6,965	6,965				586,220	586,220	8			481,835	480,667	4		4	924,798
Maine	10,383	10,383			3	363,086	363,086	2			1,861	1,861			1	285,059
Maryland	500,850	469,057	2	2	12	874,458	730,660	4		4	30,175	30,175			6	399,876
Massachusetts	346,270	335,829	1		1	791,642	790,770	5	2		763,830	763,830	2	1		1,250,554
Michigan	1,333,563	1,322,460	3	4	26	240,766	240,766	1			752,429	722,268	2		9	906,424
Minnesota	643,147	642,783	5	4	2	977,175	977,175	5	4	6	4,985	4,985			1	1,236,480
Mississippi	253,874	253,874	2	1	10	843,110	843,110	10			25,808	25,808			2	646,344
Missouri	120,702	120,702		2		2,192,393	1,736,973	8	2		203,565	203,565		2		1,480,365
Montana	141,549	141,549	2		2	99,778	99,778	1								733,051
Nebraska	297,242	296,146	4		21	1,023,081	1,023,081	19		1	13,020	13,020			3	475,292
Nevada	120,817	116,466	2		1	63,435	63,435	2		2	22,635	22,635			7	196,853
New Hampshire	265,368	265,069	4	1	3	96,565	96,240	2	1	2						313,961
New Jersey	852,812	850,349	4	1	2	622,904	497,354	3	1		354,985	295,560	1	2		941,341
New Mexico						71,000	71,000	1			259,103	252,068	3		1	519,348
New York	2,422,733	2,364,650	2	13		1,982,146	1,941,007	3	8		502,645	464,285		1		3,729,719
North Carolina	619,637	616,759	2	6	31	176,292	176,292	1	1	1	51,795	51,795			13	1,407,381
North Dakota	289,022	287,819	5			470,895	470,895	5								872,117
Ohio	1,557,247	1,542,030	8	1	12	2,752,053	2,428,331	10		1	392,870	188,740	1		2	1,715,420
Oklahoma	250,854	245,570	2		34	880,359	878,439	6		3	372,820	334,783	3		4	1,458,144
Oregon	419,535	355,255	4		3	281,502	228,715	2			13,433	13,433			4	403,396
Pennsylvania	1,907,614	1,876,905	14	1		3,016,940	3,002,411	14			359,074	359,074	2			2,526,669
Rhode Island	205,241	205,241		1		3,655	3,655									273,921
South Carolina	351,099	334,334	6		23	259,152	241,711	3	3	1	214,585	99,452		2	7	992,889
South Dakota	511,858	511,747	13		9	515,821	499,871	9			41,200	41,200	2			829,275
Tennessee	301,580	289,686	3	3		1,325,647	1,325,647	7	1		102,106	102,106		1	2	1,005,387
Texas	1,329,036	1,309,617	16			1,359,773	1,356,641	13		4	4,350	4,350			1	2,368,884
Utah	68,276	67,534	2		22	59,888	59,888	1		6	60,140	60,140			23	359,457
Vermont	17,469	16,742			4	322,869	293,090	2								114,613
Virginia	562,165	562,165	4	4	3	347,127	327,167	3	1	1	4,015	4,015			1	965,269
Washington	392,291	392,044	3	2	3	1,041,232	344,558	5			5,192	5,192			2	435,542
West Virginia	253,143	247,512	3		6	654,510	654,510	6	1	2	349,830	349,830	1		1	430,253
Wisconsin	468,542	451,283	2		39	571,419	570,431	3	2	7	12,687	12,687			2	1,674,169
Wyoming	481,187	466,531	6		1	1,974	1,974				8,416	8,416			6	430,012
District of Columbia	3,655	3,655				298,213	273,744		1							103,351
Hawaii	189,832	189,811	2			211,977	211,462	2								282,756
Puerto Rico	103,629	102,980	1			780,619	772,706	11								363,914
TOTALS	23,365,998	22,566,277	169	55	484	35,563,605	33,216,435	250	45	103	8,056,996	7,360,195	34	15	249	47,608,074

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

FEDERAL WORKS AGENCY
PUBLIC ROADS ADMINISTRATION

VOL. 23, NO. 4 ▽ JUNE 1942

OBSERVERS RECORD HIGHWAY CAPACITY DATA IN ILLINOIS

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PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

issued by the

FEDERAL WORKS AGENCY

PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 23, No. 4 June 1942

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Commissioner of Public Roads, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

RESULTS OF HIGHWAY-CAPACITY STUDIES

BY THE DIVISION OF HIGHWAY TRANSPORT, PUBLIC ROADS ADMINISTRATION

Reported by O. K. NORMANN, Highway Engineer-Economist

Preliminary results of highway capacity studies conducted by the Public Roads Administration in cooperation with the highway planning survey organizations in several States were published in 1939 and have been used in the solution of numerous traffic-control and highway-design problems by the various State highway departments and other agencies. Data obtained prior to 1939 have been supplemented by more recent data and the analyses completed to determine theoretical, possible, and practical capacities for 2-, 3-, and 4-lane highways.

Since the results are based on data gathered prior to the present tire and gasoline rationing programs, they will be of maximum value upon a return to normal driving conditions. However, many of the principles developed are applicable to military traffic as well as the efficient movement of civilian traffic in the neighborhood of defense industries.

The maximum theoretical capacity of a single traffic lane is about 2,000 vehicles per hour, occurs at speeds above 30 miles per hour, and can be attained on 4-lane highways or on short sections of 2-lane roads.

The total possible capacity of a long section of 2-lane highway with good alinement, carrying few trucks, is about 2,000 vehicles per hour or one-half its theoretical capacity. A corresponding value for 3-lane, 2-directional highways is 3,600 vehicles per hour. The maximum possible capacity of a 4-lane highway is 8,000 vehicles per hour or 4,000 vehicles per hour for the 2 lanes used by traffic traveling in the one direction. When trucks constitute 17 percent of the traffic the possible capacity of a 2-lane highway is reduced by about 25 percent.

The practical working capacity of a highway is a relative value and depends upon local conditions as well as the width or number of lanes. The maximum practical working capacities for 2-, 3-, and 4-lane rural highways under normal conditions are 800, 1,400, and 2,800 vehicles per hour, respectively. However, the results presented are of sufficient scope to be of value in the determination of the practical capacity for any highway condition and in the solution of numerous traffic control and design problems.

THE highway capacity studies conducted by the Public Roads Administration in cooperation with the highway planning surveys of several State highway departments are closely related to the studies of lateral placement, truck performance, driver behavior, and passing practices. All are part of a program designed to obtain information regarding the operation and performance of motor vehicles under a wide range of traffic and highway conditions. This information will be of practical value in facilitating safe and economical highway transportation. The ultimate objective of the highway capacity studies is to determine practical carrying capacities for all types of highway design, but in particular for modern 2-, 3-, and 4-lane roads, and to determine what effect certain highway design features, motor-vehicle characteristics, highway regulations, and driver characteristics, have on the practical capacities. To reach this objective, use must be made of the facts brought to light by the related studies, especially the passing-practice study, and complete and reliable data on traffic movement during light and heavy densities must be available for each major highway condition that is to be considered.

Some of the preliminary results based on data gathered on about 300,000 vehicles from 1934 through 1937 have been published.[1] Since then, additional studies have been conducted in Massachusetts, Illinois, and California, under traffic conditions that will yield data useful in the determination of highway capacity. Speed-placement studies have also been conducted in Iowa, Minnesota, Ohio, Oregon, South Carolina, Texas, and Washington. However, the data gathered in the latter group of States do not include exceptionally heavy traffic densities, since the studies were made primarily to obtain speed and transverse placement data on various curves, different pavement and shoulder widths, and other highway design features.

Since 1938, greatly improved equipment[2] has been used in obtaining the necessary data but during all studies the speeds and spacings from other vehicles were obtained for practically 100 percent of the vehicles on the highway. This report covers the results of an analysis of all data gathered prior to 1938 and the pertinent 1939 data.

In analyzing the data to determine practical working capacities for 2-, 3-, and 4-lane highways, numerous facts regarding the operation and movement of vehicles on various highways have been brought to light. A few will be presented. A thorough understanding of all of these in addition to the findings of the driver behavior,[3] motor-vehicle performance,[4] and passing practice studies,[5] is necessary in any scientific determination of practical capacities for particular roadway conditions.

---

[1] Preliminary Results of Highway Capacity Studies, by O. K. Normann. PUBLIC ROADS, February 1939.

[2] New Techniques in Traffic Behavior Studies, by E. H. Holmes and S. E. Reymer. PUBLIC ROADS, April 1940.

[3] A Study of Motor-Vehicle Drivers and Speed in Connecticut, by Harry R. De Silva. PUBLIC ROADS, July 1940.

[4] Hill-Climbing Ability of Motor Trucks, by Carl C. Saal. PUBLIC ROADS, May 1942.

[5] Passing Practice on Rural Highways, by C. W. Prisk. Highway Research Boar Proceedings, 1941.

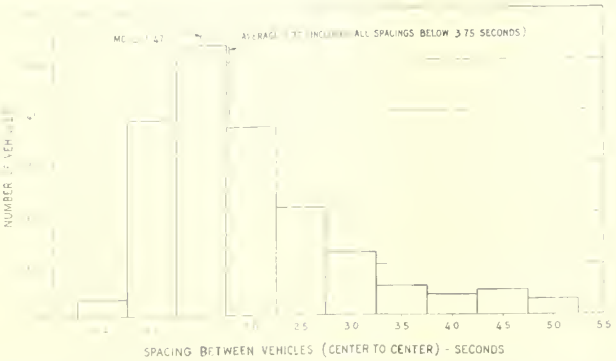

FIGURE 1. FREQUENCY DISTRIBUTION OF TIME SPACINGS BETWEEN VEHICLES TRAVELING AT THE SAME SPEED. (30-35 MILE-PER-HOUR GROUP ON 2-LANE LEVEL HIGHWAYS).

MINIMUM SPACINGS BETWEEN VEHICLES VARY WITH TYPE OF HIGHWAY AND VEHICLE SPEED

A review of published material on highway capacity [6] shows that practically all attempts to determine the maximum capacity of a highway at various speeds have been based upon safe following distances which in turn have been based upon assumed reaction times, coefficients of friction, etc. The safe following distances as calculated by different authors vary from 40 to 150 feet at 20 miles per hour and from 87 to 1,230 feet at 60 miles per hour. The maximum capacity of a single traffic lane based upon these theoretical derivations varies from 1,500 to 4,800 vehicles per hour. A few authors maintain that there is an increase in capacity with an increase in speed but the majority hold that the maximum capacity is attained at speeds in the neighborhood of 20 miles per hour or less.

The traffic data analyzed by the Public Roads Administration include information for drivers of 11,000 vehicles that were trailing another vehicle in the same traffic lane and traveling at practically the same speed. Figure 1 shows the distribution of time spacing for all vehicles on 2-lane highways that were traveling between 30 and 35 miles per hour and following another vehicle traveling at practically the same speed. Under such a condition it is expected that there will be a range in the time spacings at any particular speed for different drivers as well as for the same driver due to inability to maintain for very long the exact speed of the vehicle being followed. However, in the distribution are also included drivers who chanced to be traveling at the same speed as the preceding vehicle on the highway when their speeds and spacings were recorded but who were not at their minimum spacings. The spacing values for these drivers should not be included and therefore all spaces exceeding 3.75 seconds, which is the highest value that could be considered within the normal distribution, have been excluded in determining the average minimum spacing for the particular condition represented. In a similar manner, exceedingly long time intervals were excluded from the average values for other speeds and highway conditions and the resulting values were used to obtain figures 2 and 3.

Figure 2 shows the average minimum distance spacings allowed by drivers at different speeds. All curves are for daytime operation except the one for 2-lane highways at night. Although insufficient data were obtained to determine accurate spacings at low speeds for any but daylight conditions on 2-lane highways, all curves probably approach the one for the 2-lane highways as the speed decreases.

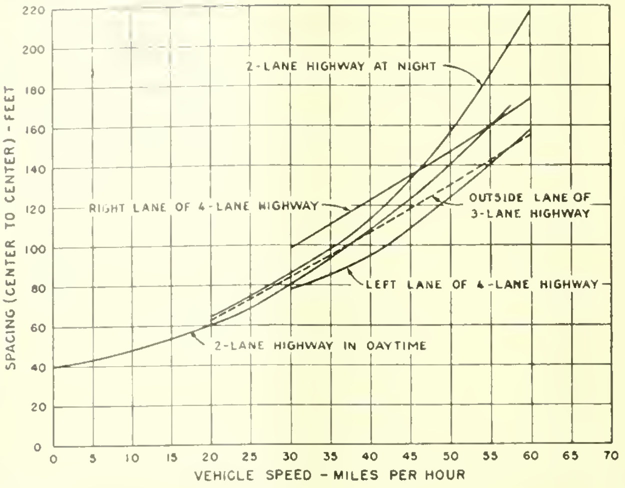

FIGURE 2. AVERAGE MINIMUM DISTANCE SPACINGS ALLOWED BY DRIVERS WHEN TRAILING ANOTHER VEHICLE.

Although all curves show an increase in the distance spacing with an increase in speed, it is evident that factors other than safe stopping distances are involved in the spacings allowed by drivers when following another vehicle. On 2-lane roads there is a relatively slight decrease in the distance spacing with a decrease in speed below 20 miles per hour. As long as both vehicles are moving at a uniform speed there is an average spacing of at least 40 feet between centers. This is considerably greater than the distance between them when the two vehicles come to a stop on the highway. One reason for this may be that it is hard to maintain a uniform spacing at low speeds. Another reason may be that at low speeds drivers do not desire to drive any closer to the vehicle ahead than the distance required to prevent the vehicle behind from passing and crowding back into line.

At night, drivers allowed a greater clearance than in the daytime for corresponding speeds. On 3-lane roads, a greater distance was allowed at the lower speeds and a shorter distance at the higher speeds than on 2-lane roads. On 4-lane highways the drivers in the left-hand lane allowed shorter spacings than did those in the right-hand lane. It is possible that the right-hand lanes contained a higher percentage of cautious drivers or that while a driver was in the left lane he allowed a shorter distance because he felt that the driver of the preceding vehicle was not likely to slow down suddenly.

A comparison was made of values obtained during 1934 and 1939 and indicated that there was no change during these years in the distance that drivers followed one another when traveling at the same speed.

Figure 3 shows for different conditions the numbers of vehicles traveling in a single traffic lane that could pass a given point if all drivers traveled at the same speed and no space between vehicles exceeded the distance allowed by the average driver while trailing another vehicle. Although the values shown by the curves are based upon observed spacings, they are "theoretical" lane capacities. The maximum theoretical lane capacity on a 2-lane road is 2,000 vehicles per hour in the daytime and about 1,800 at night, both being attained at a vehicle speed of about 33 miles per

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[6] Resume of Previously Published Material on Highway Capacity, by O. K. Normann, and Asriel Taragin. Paper presented at meeting of Highway Research Board, 1941.

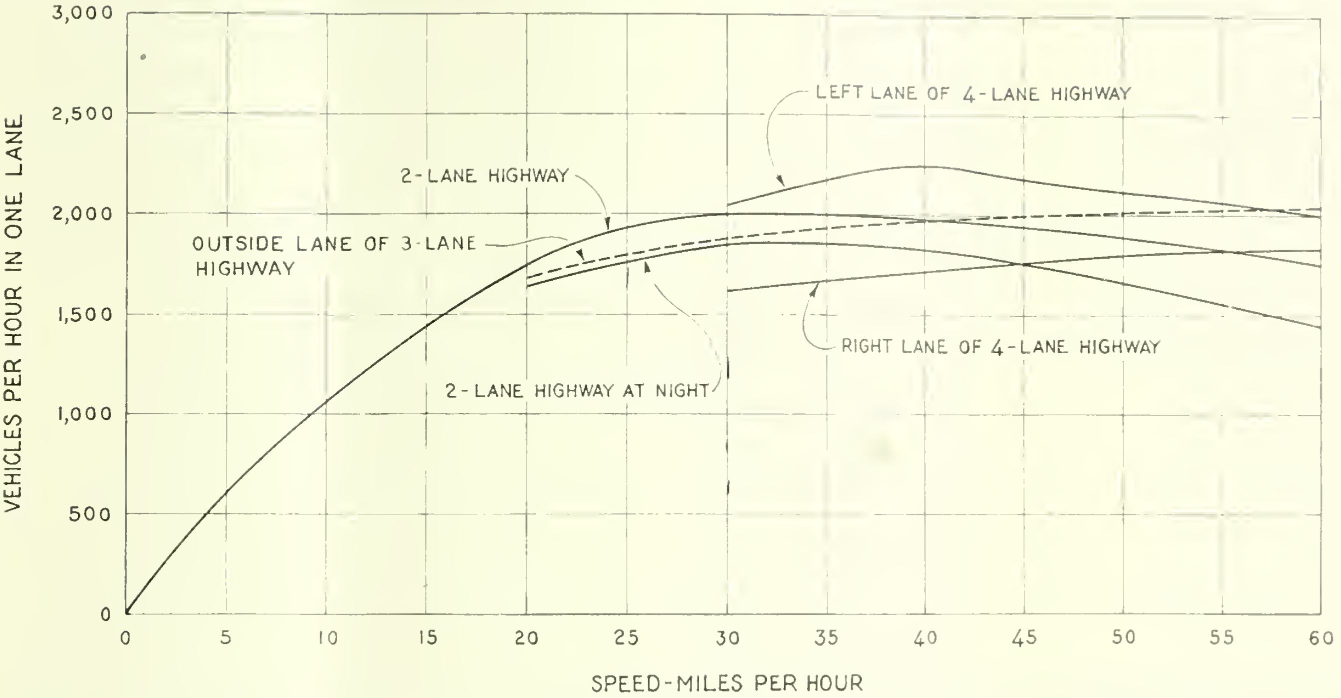

Figure 3.—Theoretical Maximum Traffic Capacity With All Vehicles Traveling at the Same Speed (Based on Average Spacings).

hour. The left lanes of 4-lane highways reach their maximum theoretical capacities of about 2,250 vehicles per hour when the vehicles travel at a speed of 40 miles per hour, while the right-hand lanes and the lanes of 3-lane highways show increased capacities with increased speeds within the range of speeds observed. The values for 3-lane highways refer to the outside lanes. These theoretical lane capacities may be approached over very short sections of highway that act as bottlenecks and where drivers expect to stay in line. However, since all vehicles must stay in line and travel at the same speed, the speed of the slowest moving vehicles in such a case will determine the actual capacity.

MEAN DIFFERENCE IN SPEED BETWEEN SUCCESSIVE VEHICLES BEST INDEX OF POSSIBLE HIGHWAY CAPACITY

Probably the most important conclusion that can be drawn from the results of this analysis based on actual spacings as compared with the results based on assumed or calculated values is that maximum capacities occur at speeds well above 20 miles per hour. If conditions are such that traffic at bottlenecks can be kept moving at speeds of 20 to 25 miles per hour, one lane will handle nearly twice as many vehicles per hour as can be handled if traffic is required to slow down to 10 miles per hour. This is a very important consideration when regulating traffic under emergency conditions such as at construction projects or where an accident has occurred and vehicles that normally use 2 lanes are required to use one lane.

Under actual highway conditions all drivers do not desire to travel at the same speed. To permit drivers to travel at their desired speeds it must be possible for the drivers traveling at the higher speeds to pass those traveling at the slower speeds. Obviously, on a straight, level 2-lane highway this becomes increasingly difficult as the traffic density increases until finally the faster drivers are forced to stay in line and travel at a reduced speed. As the number of opportunities to pass slow-moving vehicles decreases, the highway becomes more and more congested.

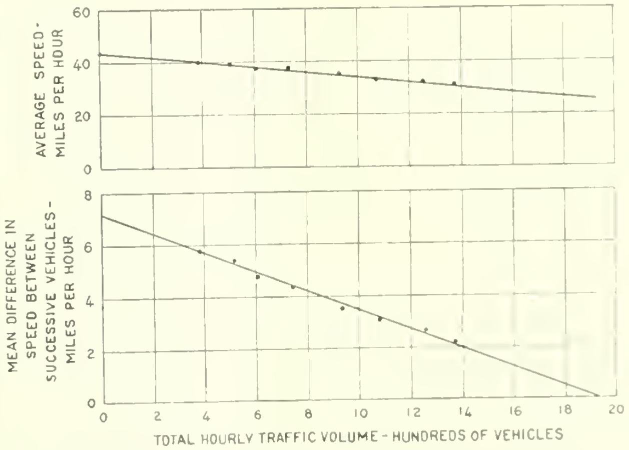

Figure 4.—Speed and Mean Difference in Speed for 2-Lane Level Tangent Highway Section in Illinois.

Efforts to find a measure of the congestion on a highway have revealed that the mean difference in speed between succeeding vehicles is the best index available. Many other indices, including the drop in speed with increased densities, were tried and found lacking.

Figure 4 shows both the average speed and mean difference in speed for various traffic densities on a 2-lane road in Illinois. At the light traffic densities, when all drivers could travel at their desired speed, the average speed on this 2-lane highway was 43 miles per hour and the mean difference in speed between succeeding vehicles was 7.2 miles per hour. As the traffic density increased there was a gradual decrease in the average speed and also in the mean difference in speed, until at a density of 1,380 vehicles per hour the mean difference in speed was only 2.4 miles per hour, although the average speed had only decreased to 31 miles per hour.

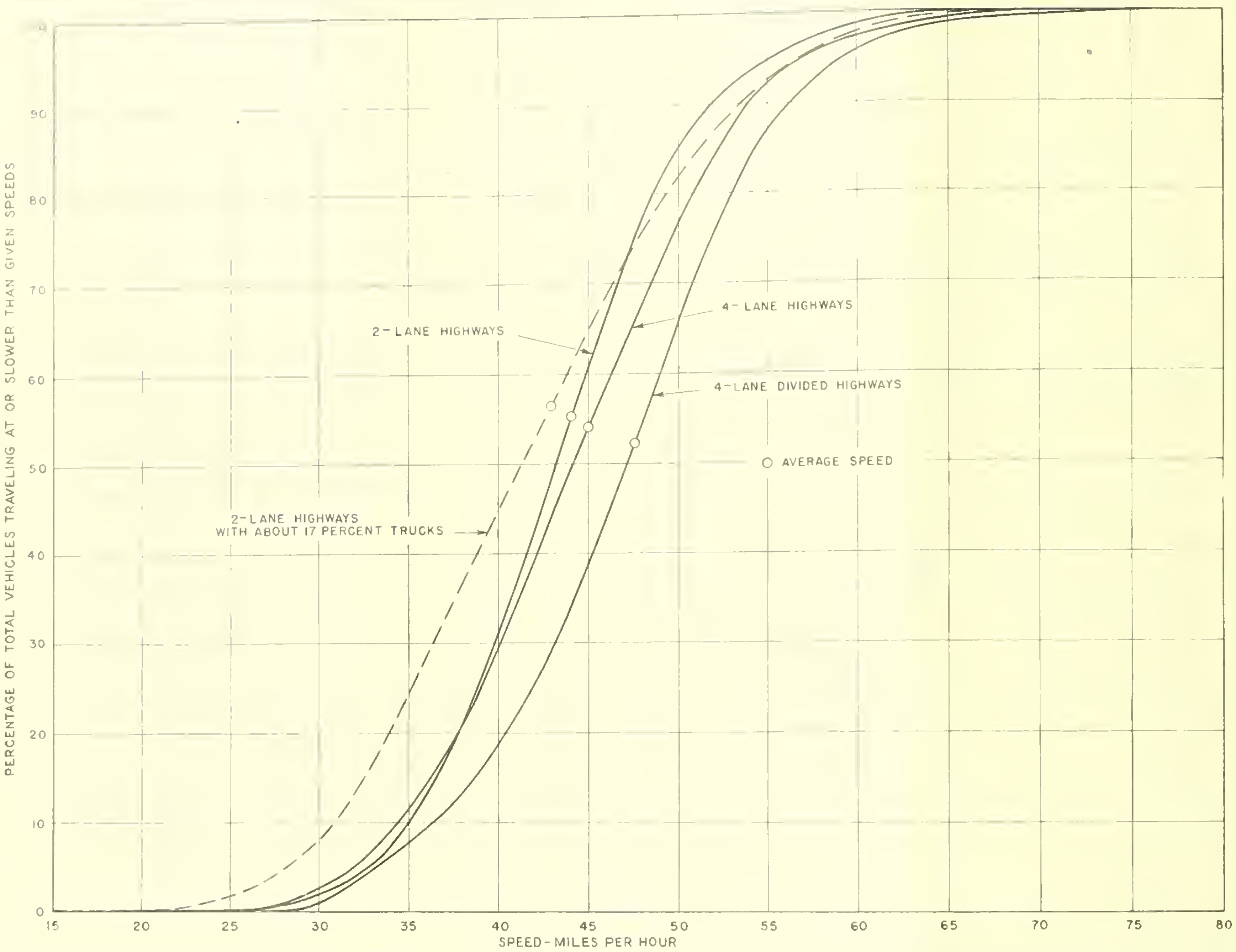

Figure 5.—Cumulative Frequency Distribution of Free Speeds on Level Tangent Sections of Illinois Highways Included in Capacity Studies.

Since it was evident that for all practical purposes the relationship was a straight line, the line for the mean difference in speed was extended until it intersected zero difference in speed at 1,940 vehicles per hour.

It is obvious that when there is a zero difference in speed between succeeding vehicles, no passings can be made and no driver can travel faster than the vehicle immediately ahead. The result is that groups of cars are formed, each following a slow-moving vehicle. If the highway is long enough, one group may catch up to another group, but there is no possibility of filling the spaces between groups by one group passing the other. When this condition occurs, the possible capacity of the highway has been reached, and, although vehicles from side roads may tend to fill the gaps, they cannot travel faster than the group of vehicles ahead. Possible capacities as used in this report, therefore, refer to the number of vehicles per hour that can travel over long stretches of highway that are free from intersections that would further reduce the roadway capacities.

On this particular section of highway, the maximum possible capacity of 1,940 vehicles per hour would be reached when the average speed had decreased to 26 miles per hour. With traffic moving at 26 miles per hour, the highway would not ordinarily be considered completely congested, but the mean difference in speed is a gage that measures the congestion on a highway and shows when the possible capacity is reached. When it has decreased to zero, even a slight additional load or some other factor may cuase a complete tie-up of traffic or an immediate drop in speed to a value below the speed at the possible capacity, which in this case is 26 miles per hour.

The Illinois studies included data for a number of 2-lane, 4-lane undivided, and 4-lane divided highways carrying similar traffic and comparable in all respects except for the number of traffic lanes. At some of the locations studied traffic went directly from one type of highway to another. Figure 5 shows the distribution of speeds and the average speed for vehicles traveling on each type of highway during very low traffic densities when each driver was free from any restriction in speed by other vehicles. The curves look very similar, but if studied closely it may be seen that the data represented by any one are quite different from those for any of the others.

On the 4-lane divided highway the average speed of the free-moving vehicles was 47.5 miles per hour, which is the highest average speed yet recorded in these studies. As far as the highway itself was concerned, there was no reason why all vehicles could not have traveled 70 miles per hour or faster, as the highway was designed for speeds of 100 miles per hour, and has no major intersections at grade. Yet with the study loca-

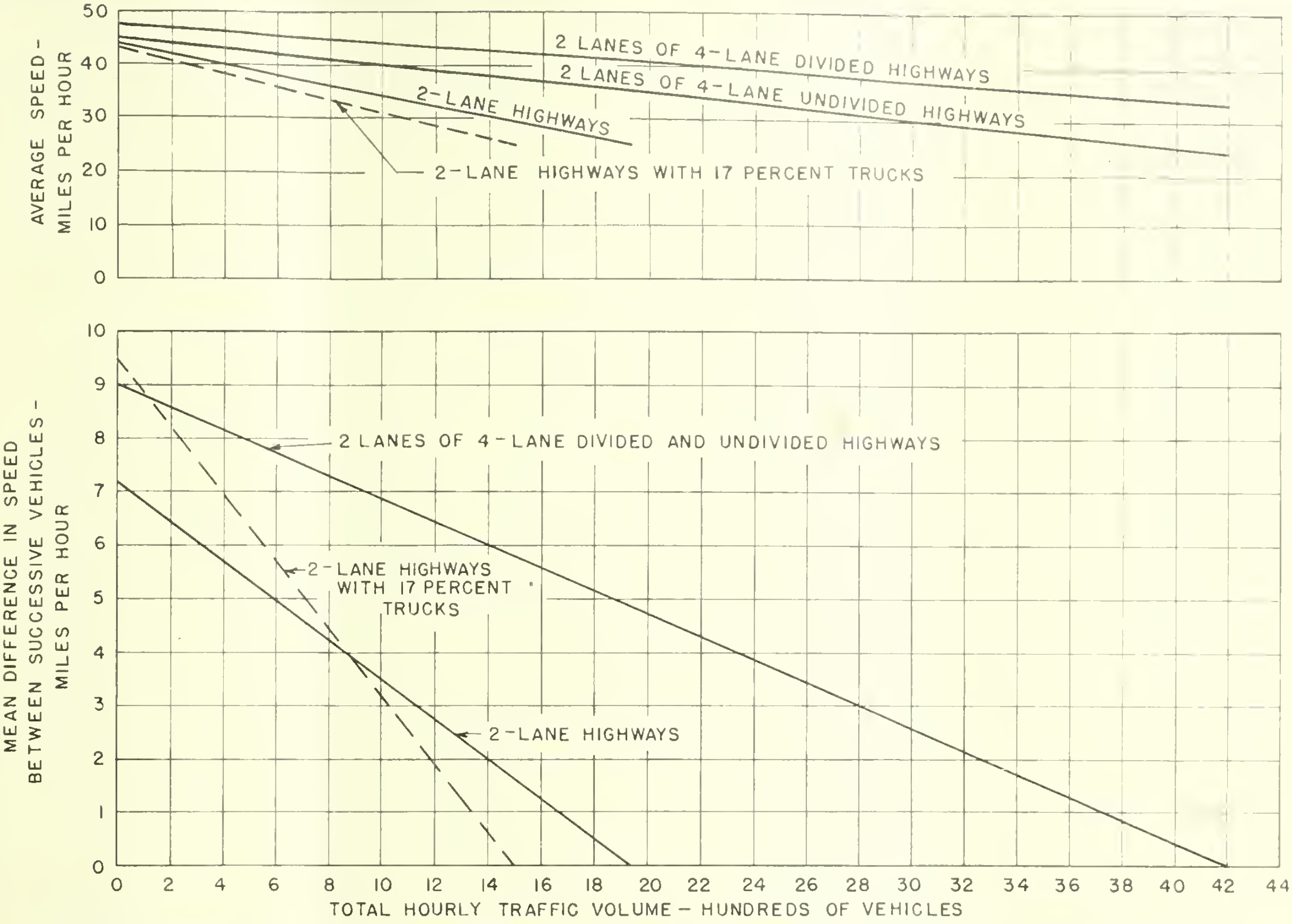

Figure 6.—Speeds and Speed Differences on Comparable Level Tangent Highway Sections in Illinois.

tion on this highway at the center of a 5-mile level tangent, 18 percent of the drivers desired to travel less than 40-miles per hour, 80 percent desired to travel less than 53 miles per hour, and only 5 percent traveled over 60 miles per hour.

The average speed for free-moving vehicles on the 4-lane undivided highway was 45 miles per hour and on the 2-lane highway it was 43 miles per hour. All of these highways carried few trucks during the study periods. The 2-lane highways represented by the dashed curve carried about 17 percent trucks and, as a result, the speed range was greater. Only 20 percent of the vehicles traveled between 40 and 45 miles per hour, compared with nearly 30 percent within the same range on the 2-lane highways carrying a negligible number of trucks.

When the average speed and mean difference in speed between successive vehicles for the various traffic volumes were plotted for each highway condition, figure 6, based upon reliable and detailed data for 81,581 vehicles, was obtained. The 2-lane highways with few trucks had possible capacities of 1,940 vehicles per hour at an average speed of 26 miles per hour. The same type of highway carrying 17 percent trucks had a possible capacity of 1,500 vehicles per hour at a speed of 26 miles per hour.

Two lanes of the 4-lane highways had total possible capacities of 4,200 vehicles per hour, this value occurring at a speed of 24 miles per hour on the undivided sections and at 33 miles per hour on the divided sections.

The points at which the lines for the speed differences intersect the Y-axis show the mean difference in speed when the vehicles could move freely and pass as many other vehicles as was necessary for the drivers to maintain their desired speeds. On the 2-lane highways the average difference in speed was 7.2 miles per hour. When there were 17 percent trucks, the difference was 9.5 miles per hour, due to the wider range in speeds when trucks were intermixed with passenger cars. On the 4-lane highways, during free movement there was a mean difference in speed of 9 miles per hour, which is higher than for the 2-lane highways without trucks. This was caused by the drivers' desire to travel at a wider range of speeds on the 4-lane than on the 2-lane highways.

The capacity studies in New York and Massachusetts were made under a wider range of highway conditions than were those in Illinois. When data from all the locations on level tangent sections of highways were classified by the number of lanes and the speed at which the vehicles were traveling during very light traffic densities, they fell into seven different groups, as shown by figure 7.

AVERAGE VEHICLE SPEEDS VARY WIDELY

The speeds of the free-moving vehicles on 2-lane highways are represented by curves A, B, and C. Curve A shows the frequency distribution of speeds through the residential section of a small town on a highway having no cross traffic and where the two

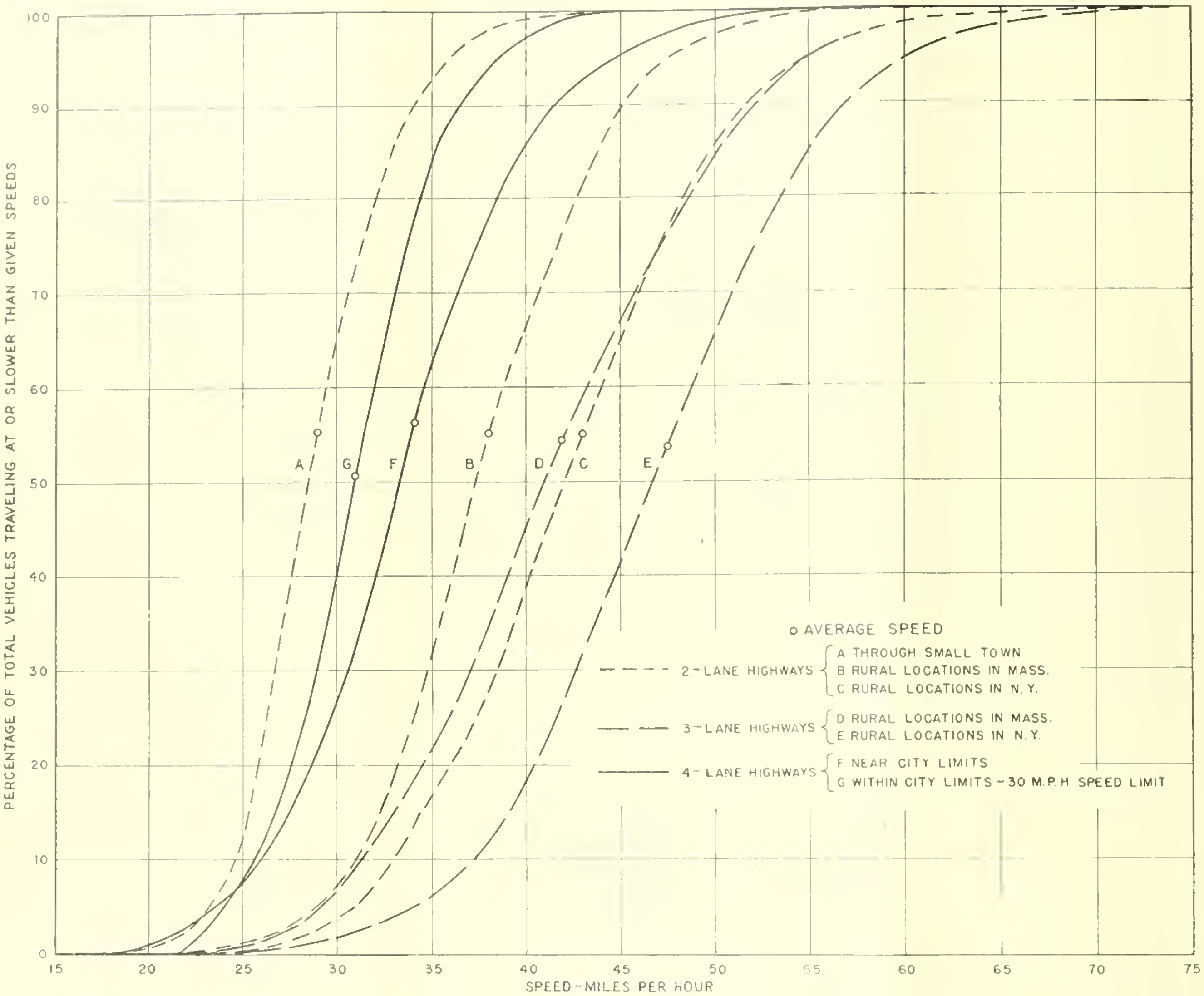

Figure 7. Cumulative Frequency Distribution of Free Speeds on Level Tangent Highway Sections in New York and Massachusetts.

lanes were free from parked vehicles. At this location, drivers uninfluenced by other traffic traveled from 16 to 45 miles per hour, with 80 percent traveling between 25 and 35 miles per hour. Curve B represents 2-lane highways at three rural locations in Massachusetts, while curve C represents highways in three rural locations in New York.

At each of the Massachusetts locations the average speed for free-moving vehicles was 38 miles per hour, while at the New York locations it was 43 miles per hour. The highways studied do not represent average conditions in each State, since the particular locations were selected to obtain desired data for these capacity studies rather than to make comparisons between highways in different States. Although curves B and C both represent level, tangent, rural locations, the difference of 5 miles per hour in the average free speeds may have been caused by a number of factors, such as the general alinement of the highways, the smoothness of the surface, and the distance between towns.

On each of three 3-lane highways in one group, the average free speed was 42 miles per hour, as represented by curve D, and for another group of two 3-lane highways, as represented by curve E, the average speed was 47.5 miles per hour. Curve F represents the free speeds on a 4-lane highway near the city limits of a town, while curve G represents the free speeds within the city limits where there was a 30-mile-per-hour speed limit. Thus it may be seen that even on level, tangent sections of highway unaffected by intersections or by other traffic, there is a wide range in the average free speeds at different locations.

When the average speeds and mean differences in speed are plotted for different traffic densities for these seven highway conditions, the results illustrated by figure 8 are obtained, based on data for 131,340 vehicles.

The speed and difference in speed values shown for the points where the curves intersect the line representing 0 volume are in effect values for densities approaching 0 vehicles per hour, or for drivers whose speeds are not influenced by other traffic.

Considering first the 2-lane highways (fig. 8), the average speeds on sections A, B, and C at light traffic volumes, or the average free speeds, were 29, 38, and 43 miles per hour, respectively. When all drivers could maintain their desired speeds there was a speed difference of 4.1 miles per hour on section A, 5.7 miles per hour on section B, and 8.7 miles per hour on section

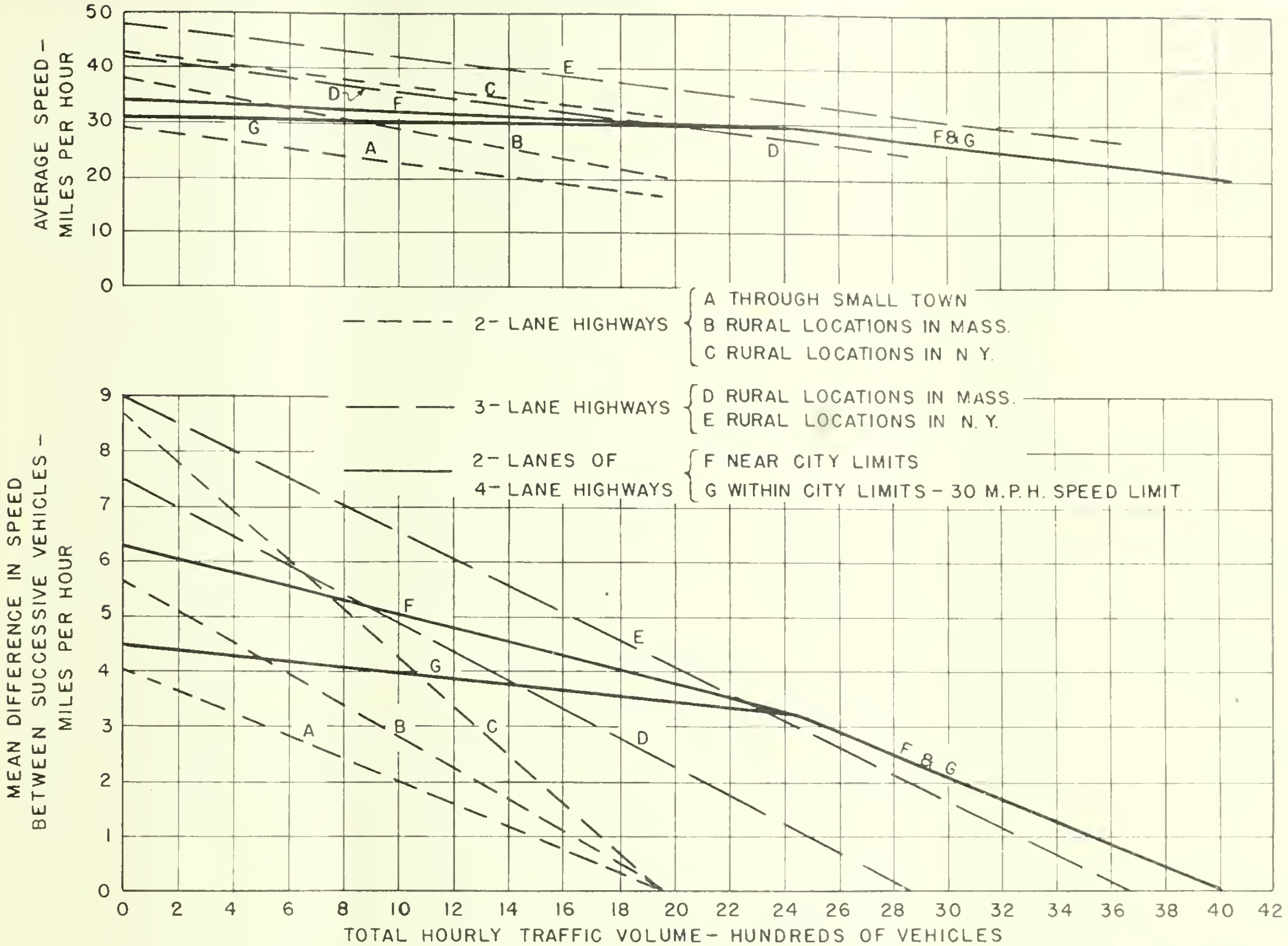

Figure 8.—Speeds and Speed Differences on Level Tangent Highway Sections in New York and Massachusetts.

C. With a given increase in traffic density, there was a greater decrease in the speed difference on section C than on section A or B. The speed difference became zero at about 1,950 vehicles per hour on sections A and C, and at 1,980 vehicles on section B. The possible capacities are practically the same for conditions represented by curves A, B, and C, although they occur at average speeds of 16.5, 20, and 31 miles per hour, respectively. The possible capacity of the Illinois 2-lane section with few trucks was 1,940 vehicles per hour at an average speed of 25 miles per hour.

Each of the 3-lane sections represented by curve D had a possible capacity of 2,860 vehicles per hour at 24 miles per hour, while each of the 3-lane sections represented by curve E had a possible capacity of 3,660 vehicles per hour at 26 miles per hour.

The speed restrictions on the 4-lane sections F and G caused lower speeds and speed differences at low volumes, but the speed restrictions apparently had no effect when the traffic volume in one direction reached 2,450 vehicles per hour. The possible capacity in both cases was about 4,000 vehicles per hour at a speed of 20 miles per hour.

From the studies in Illinois, Massachusetts, and New York, it is evident that:

1. Two-lane, level, tangent sections of highway carrying few trucks have a possible capacity of nearly 2,000 vehicles per hour. With 17 percent trucks their capacity is reduced about 25 percent.

2. Two lanes of level tangent sections of 4-lane highway carrying few trucks have a possible capacity of about 4,000 vehicles per hour. If the possible capacity of a 4-lane highway is considered to be reached when the lanes in one direction are carrying their maximum possible capacity, the possible capacity of the entire roadway depends upon the relative number of vehicles traveling in each direction. With all traffic in one direction during peak periods, the total possible capacity of a 4-lane road (retaining the two-directional character of the road) is 4,000 vehicles per hour; with traffic equally divided by direction, the total possible capacity becomes 8,000 vehicles per hour. However, since about two-thirds of the traffic is usually in one direction on most highways during peak periods, the lanes in one direction will be carrying their maximum possible capacity with a total volume of 6,000 vehicles per hour.

3. Three-lane level tangent sections of highway apparently do not all have the same maximum possible capacities, as is the case for 2- and 4-lane highways. A possible capacity of 2,800 vehicles per hour was obtained for one group of 3-lane highways, and a possible capacity of 3,600 vehicles per hour was obtained for another group. The latter group included highway sections with exceptionally long sight distances, smooth surfaces on all three lanes, and lanes that were well defined by painted lines. These factors all had a tendency to encourage the use of the center lane for passing purposes. The best 3-lane roads can therefore

be expected to have maximum possible capacities of 3,600 vehicles per hour. Since the capacity of each of the other 3-lane roads was only 2,800 vehicles per hour, there apparently is a tendency for the capacity to decrease rapidly with a lowering of the design standards.

4. The speed at which vehicles will travel when the possible capacity of a highway is reached will depend entirely upon the speed at which the slowest vehicles on the highway are traveling. Usually this will be about the same as the speed of the slowest group of vehicles during light traffic densities.

POSSIBLE CAPACITY OF 2-LANE HIGHWAY ABOUT HALF OF THEORETICAL CAPACITY

It is interesting to compare the possible capacities with the theoretical capacities. For a 2-lane highway, the possible capacity is about half of the theoretical capacity. With practically all traffic traveling in one direction on a 2-lane highway the lane normally used by oncoming traffic provides the means for passing the slow-moving vehicles so that the exceedingly long spaces ahead of slow-moving vehicles may be filled. However, one oncoming vehicle will require all other vehicles to crowd into their own lane, making the total possible capacity equal to the theoretical capacity of one lane. With traffic equally divided by direction it will be shown by figures to be presented later that few spaces sufficiently long to permit a passing exist when there are over 1,000 vehicles per hour traveling in each direction, but the drivers do not take advantage of these passing opportunities.

On 3-lane tangent highways carrying practically all traffic in one direction and two lanes filled, the possible capacity of all three lanes cannot exceed the theoretical capacity of two lanes. With traffic evenly divided, the number of passings required to keep both outside lanes full apparently cannot be made in the one center lane.

On 4-lane highways, the possible capacities are very close to the theoretical capacities. Both lanes can be used to carry traffic and at the same time provide a means of passing slow-moving vehicles so that the spaces that occur ahead of the slow vehicles can be filled.

When a highway is carrying its maximum possible capacity, no vehicle can travel at an average speed in excess of the speed of the slowest vehicle on the highway. It has been shown that all drivers do not desire to travel at the same speed even under identical highway conditions and that the speed of the slowest vehicle is generally far below the speed at which a great majority of the drivers desire to travel or should be expected to travel. It is therefore desirable to establish practical capacities for 2-, 3-, and 4-lane highways based upon the maximum interference between vehicles, or the maximum congestion, that the drivers can reasonably be expected to tolerate.

The figures that have been presented show that for any particular highway location there is no sudden drop in speed or sudden increase in interference between vehicles with an increase in the traffic density until the possible capacity of the highway is reached. Rather, the opportunity for each individual driver to travel at his desired speed is gradually reduced as the density increases. The standard of practical capacity adopted for a highway is therefore a relative value that will vary even for highways of the same design and will depend upon:

1. The particular type of traffic the highway serves,
2. The congestion that will be tolerated by drivers in various sections of the country and localities under certain conditions, and
3. Economic considerations such as the funds available for highway construction and congestion reduction.

In addition to figures already presented showing the relative congestion at different traffic volumes on tangent sections of highway, other information regarding driver behavior on 2-, 3-, and 4-lane tangent sections that is useful in arriving at practical highway capacities has been obtained from the study data. Such information includes the number of passings that occur, and the distribution of time spacings between vehicles, the distribution of speeds at various traffic densities, and for 3- and 4-lane highways the number and speed of vehicles traveling in the inside and outside lanes.

Figure 9 shows the percentage of vehicles, moving in one direction on 4-lane level tangent rural highways, that use the right-hand and left-hand (outside and inside) lanes during a wide range of hourly traffic volumes. Data for a total of 102,807 vehicles obtained on six undivided 4-lane highway sections and five divided sections were analyzed to construct this figure. All highways had smooth surfaces with lane widths as shown in table 1. On all roads during the studies there were relatively few trucks, the number usually not exceeding 5 percent of the total traffic.

With the exception of the one undivided highway with 9-foot lanes, there was no marked difference at the same traffic density in the number of vehicles using the respective lanes on highways with different lane widths. Although more vehicles used the outside lanes on the divided than on the undivided highways, the difference was slight and occurred only at the low hourly traffic volumes. Therefore, for all practical purposes the results as shown by figure 9 may be considered as representative of all 4-lane rural highways in connection with any calculations with respect to capacities.

TABLE 1.—*Width of lanes on 4-lane undivided and divided highways on which distribution of vehicles between traffic lanes was studied*

4-LANE UNDIVIDED HIGHWAYS

Number of highway sections included	Lane widths	
	Outside	Inside
	Feet	*Feet*
1	9	9
1	9	11
3	10	10
1	12	13

4-LANE DIVIDED HIGHWAYS

Number of highway sections included	Outside	Inside
2	10	10
2	10	11
1	11	11

At traffic volumes up to 1,700 vehicles per hour for both lanes more vehicles traveled in the right-hand lane than in the left-hand lane. Above this traffic volume a majority of the drivers chose to travel in the left or inside lane until, at a volume of 3,200 vehicles per hour, 60 percent, or 1,920 vehicles, were in the left lane. Since this value approaches the maximum theoretical capacity of the left lane (fig. 3) the right lane must carry a larger proportion of the total number of vehicles during volumes above 3,200 vehicles per hour. The difference between the relative number in each lane is considerably amplified at the low traffic

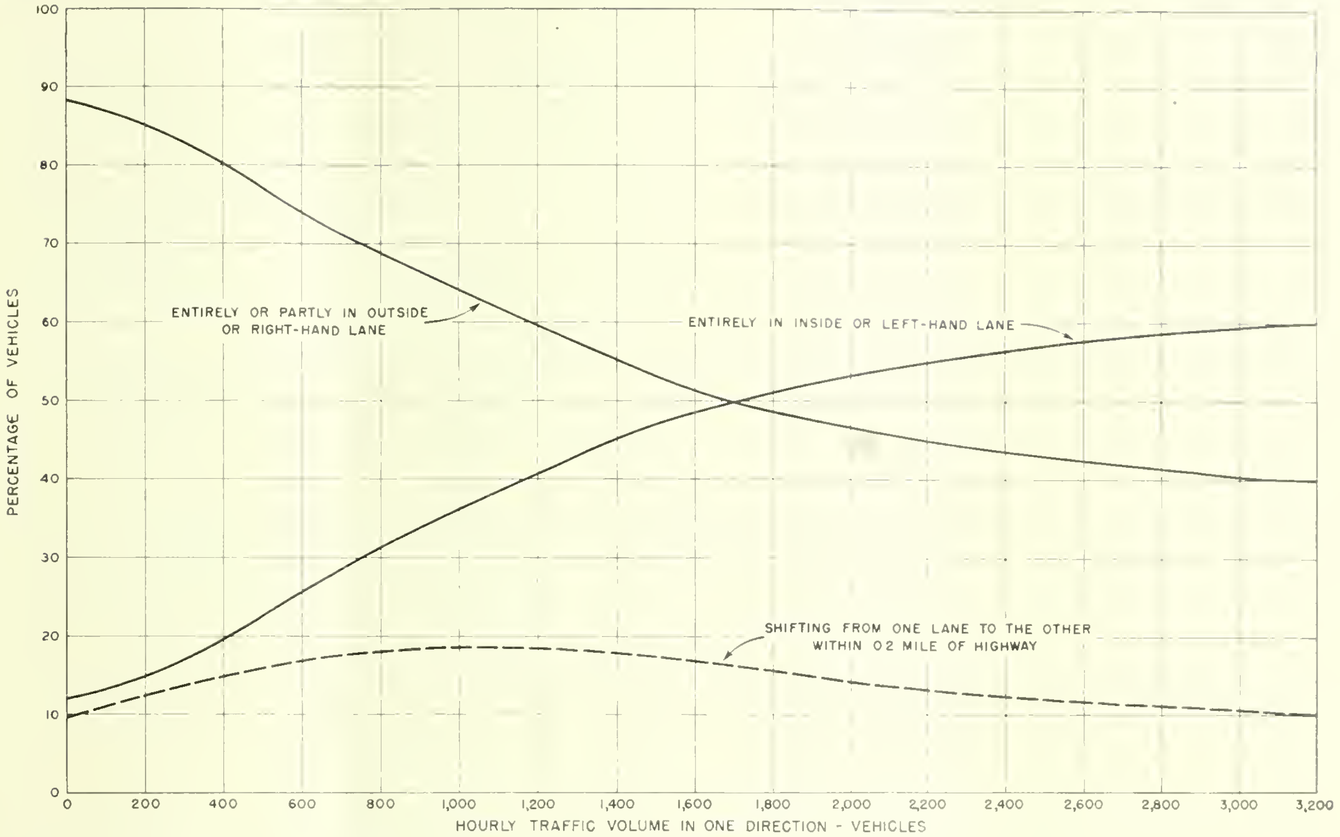

FIGURE 9.—DISTRIBUTION OF VEHICLES BETWEEN TRAFFIC LANES ON 4-LANE HIGHWAYS DURING VARIOUS HOURLY TRAFFIC VOLUMES.

densities by including the vehicles straddling the lane line with those in the right-hand lane. The data for most of the study did not permit vehicles straddling the lane line to be segregated. However, on two of the more recent studies transverse placements to the nearest foot were obtained for all vehicles. At low traffic densities about 17 percent of the vehicles on an undivided highway were straddling as against 12 percent on a divided highway.

The lower curve (fig. 9) shows the percentage of vehicles that shifted from one lane to the other in a section of highway 0.2 mile long. The maximum occurs at a volume of 1,000 vehicles when, on an average, all vehicles went from one lane to the other within 1.1 miles of straight level highway.

It can reasonably be expected that if there were no congestion or interference between vehicles, the percentage of vehicles that would shift from one lane to the other would increase or at least would not decrease with an increase in traffic density. When the traffic volume exceeded 1,100 vehicles per hour in one direction, the vehicles were not moving freely since at this density the percentage of vehicles that shifted from one lane to the other started to decrease. However, the interference between vehicles probably was not great enough at this density to consider the highway congested.

The results shown by this figure illustrate the futility of basing highway design on oil patterns. On practically all highways at least 95 percent of the total travel is performed during periods when traffic is relatively light as compared to the volumes a highway must be designed to handle without being considered congested. Some design features, such as acceleration and deceleration lanes, are designed mainly to be of service during high traffic densities. Oil patterns, which represent primarily low traffic densities, are not a true index of their value.

As expected, the average speed for vehicles traveling in the left-hand lane on 4-lane divided or undivided highways was always higher than for vehicles in the right-hand lane. There was also a tendency for the difference in speed, as well as the percentage difference, to be greater for roads where the average speed was high than for roads where the average speed was low. On highways where the average speed was about 32 miles per hour at low densities, the average speeds in the left-hand lanes were from 11 to 16 percent higher than those for the right-hand lanes, while on sections with an average free speed of about 45 miles per hour the difference varied from 15 to 22 percent.

RELATIVELY LITTLE TRAFFIC CARRIED BY CENTER LANE OF 3-LANE HIGHWAY

There was no apparent tendency for the difference in average speed on the lanes to decrease uniformly as the traffic density increased except at the very high hourly volumes. Figures 10 and 11 show the distribution of vehicle speeds between lanes at five of the study locations during the lowest and highest traffic densities during the study. The distribution at an intermediate traffic volume is also shown for study section 1 E, the only section where there was a speed limit. Although a majority of the low-speed vehicles travel in the right lane and a majority of the high speed vehicles travel in the left lane, speed distributions by lanes show that there is not the segregation of speeds between lanes

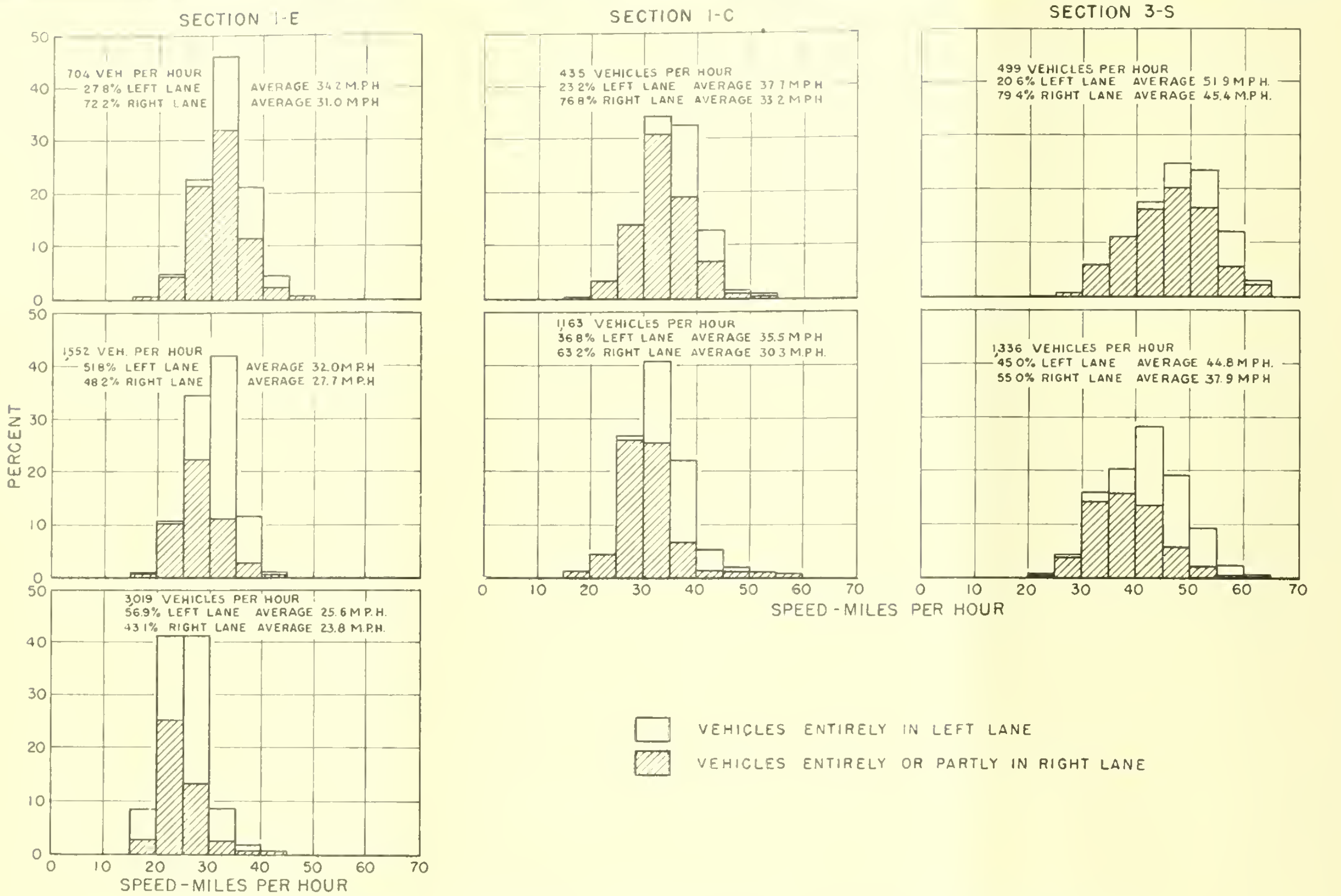

Figure 10.—Distribution of Vehicles by Speed Groups on 4-Lane Undivided Highways.

that one would expect. Slow-moving vehicles use the left lane to pass vehicles traveling at even slower speeds and passings are made to the right when more convenient than to the left. It is not uncommon to find from 13 to 19 percent of the passings being made to the right of the passed vehicle.

Three-lane, two-directional highways are constructed to accommodate traffic volumes in excess of those that a 2-lane highway will handle efficiently but that are not great enough to require a 4-lane road. To obtain a better understanding of driver behavior on 3-lane highways under various total traffic densities and directional distributions, data from studies made in 1939 on sections of 3-lane highways having 30-foot surfaces (three 10-foot lanes), level tangent alinements, and other design details which permitted high speed operation, have been analyzed (table 2). Since the sections were located in a State where 3-lane construction is common practice, most of the drivers were familiar with the operation of vehicles on 3-lane roads.

It is commonly assumed that a 3-lane highway is most efficient for locations where at least two-thirds of the traffic travels in one direction during the high-volume periods. However, it is the rule, rather than the exception, that at least twice as many vehicles will be traveling in one direction as in the other on any highway, especially during periods in which the highest densities occur.

Figure 12, based on data for 12,119 vehicles, shows that under these conditions on level tangent sections of highway the percentage of the total vehicles traveling in the center lane at any one point increases as the total volume increases to 1,500 vehicles per hour. At 1,500 vehicles per hour, only 15.9 percent were in the center lane, 13.8 percent traveling in one direction, and 2.1 percent in the other. Indications are that a maximum of about 300 vehicles per hour, or 15 percent, will be traveling in the center lane at any one point on the highway when the total hourly volume reaches 2,000 vehicles. This fact will be a surprise to anyone who has thought that about one-third of the total traffic would use each lane, that the vehicles in the center lane would consist almost entirely of vehicles overtaking the slow-moving vehicles traveling in the direction of the heavy density, and that these circumstances would be the reason for a most efficient operation of a 3-lane road when two-thirds of the traffic is in one direction.

Figure 13 shows the average speeds for the vehicles in the center and outside lanes at different traffic densities. The average speeds for vehicles in the center lane remained practically constant, while for vehicles in the right-hand lanes there was a marked decrease in speed with an increase in the total traffic volume. Even at rather high densities the average speed of vehicles while using the center lane for passing was nearly as high as the average speeds for all vehicles on high-speed, 4-lane highways during low traffic volumes. The high speeds would certainly seem to be far more hazardous on the 3-lane road, where there was always the possibility of oncoming traffic entering the same lane, than on a 4-lane road.

Figure 14 shows the distribution of speeds by lanes and direction at both a low and high total traffic density. At the higher traffic density a much larger propor-

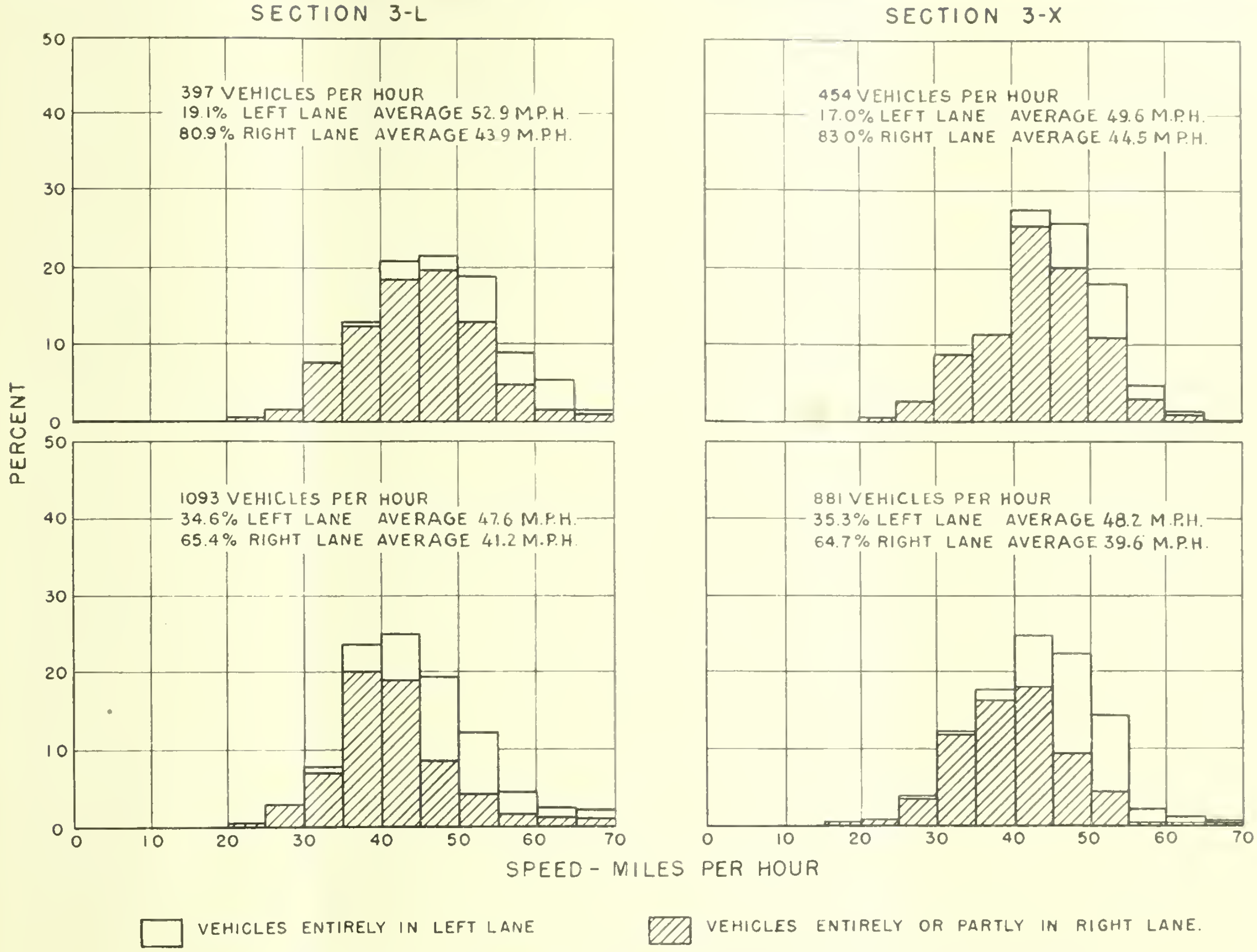

FIGURE 11.—DISTRIBUTION OF VEHICLES BY SPEED GROUPS ON 4-LANE DIVIDED HIGHWAYS.

TABLE 2.—*Distribution of vehicles between lanes and average speeds on 3-lane highway during various traffic densities while about two thirds of all traffic traveled in one direction*

	342 vehicles per hour			470 vehicles per hour			694 vehicles per hour			899 vehicles per hour			1,072 vehicles per hour			1,338 vehicles per hour			1,517 vehicles per hour			1,728 vehicles per hour		
	Vehicles			Vehicles			Vehicles			Vehicles			Vehicles			Vehicles			Vehicles			Vehicles		
	Number per hour	Percent	Average speed	Number per hour	Percent	Average speed	Number per hour	Percent	Average speed	Number per hour	Percent	Average speed	Number per hour	Percent	Average speed	Number per hour	Percent	Average speed	Number per hour	Percent	Average speed	Number per hour	Percent	Average speed
Traffic in direction of lesser density:																								
Right lane	114	33.3	35.0	185	39.4	38.7	254	36.6	38.0	224	24.9	37.5	347	32.4	36.8	398	29.8	38.0	447	29.4	37.8	464	26.9	35.4
Center lane	3	1.0	42.3	4	.8	41.6	14	2.0	45.9	15	1.7	44.3	20	1.8	41.9	31	2.3	44.0	31	2.1	46.2	23	1.3	49.6
Total	117	34.3	35.2	189	40.2	38.8	268	38.6	38.4	239	26.6	38.5	367	34.2	37.1	429	32.1	38.5	478	31.5	38.4	487	28.2	36.1
Traffic in direction of greater density:																								
Right lane	214	62.6	37.8	259	55.1	38.9	380	54.8	39.6	560	62.3	39.0	610	56.9	34.9	750	56.1	34.4	830	54.7	35.0	1,000	57.9	32.7
Center lane	11	3.1	46.0	22	4.7	42.9	46	6.6	43.6	100	11.1	42.5	95	8.9	44.2	159	11.8	44.8	209	13.8	43.0	241	13.9	44.6
Total	225	65.7	38.2	281	59.8	39.4	426	61.4	40.0	660	73.4	39.5	705	65.8	36.2	909	67.9	36.3	1,039	68.5	36.7	1,241	71.8	35.1
All traffic:																								
Center lane	14	4.1	45.2	26	5.5	42.7	60	8.6	44.1	115	12.8	42.7	115	10.7	43.7	190	14.1	44.7	240	15.9	43.4	264	15.2	45.0
Right lanes	328	95.9	37.0	444	94.5	39.2	634	91.4	39.1	784	87.2	38.6	957	89.3	34.5	1,148	85.9	35.6	1,277	84.1	36.0	1,464	84.8	33.6
Total	342	100.0	37.2	470	100.0	39.4	694	100.0	39.4	899	100.0	39.1	1,072	100.0	35.5	1,338	100.0	37.0	1,517	100.0	37.2	1,728	100.0	35.4

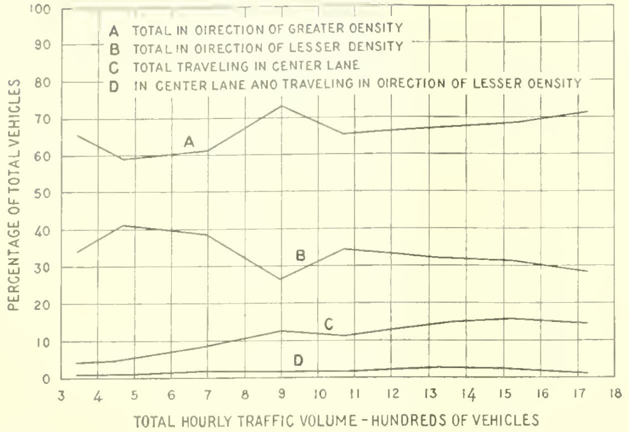

Figure 12.—Distribution of Vehicles Between Lanes on 3-Lane Level Tangent Highway.

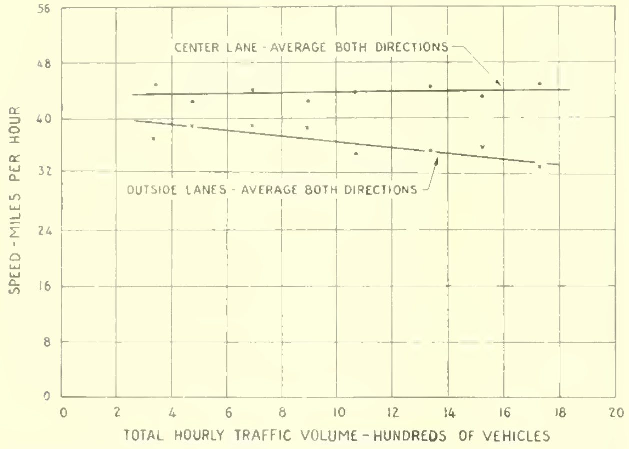

Figure 13.—Vehicle Speeds in Center and Outside Lanes on 3-Lane Highways.

tion of the high-speed vehicles was in the center lane. Although only 15.2 percent of the vehicles were in the center lane, 41.7 percent of those traveling over 40 miles per hour, 58.3 percent of those over 45 miles per hour, and 72.7 percent of those traveling over 50 miles per hour were in the center lane. At the heavy density no vehicle traveling less than 30 miles per hour was in the center lane. Apparently the vehicles had to complete the passing maneuvers and return to the right-hand lane as quickly as possible. The decreased speed of the vehicles traveling in the right-hand lane and in the direction of the heavier traffic movement was the main reason the average speed was lower during the higher total traffic volume.

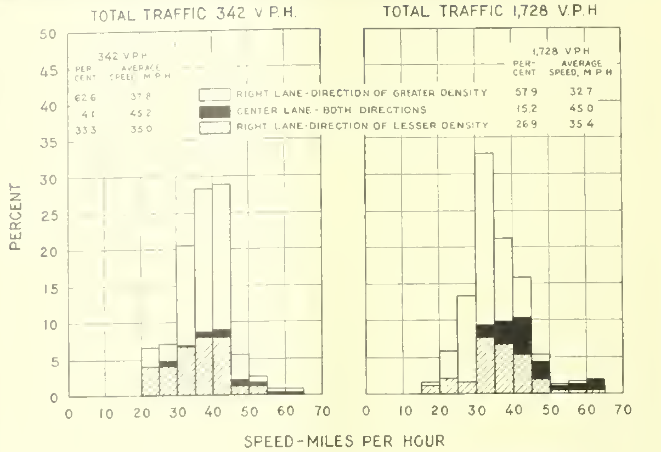

Figure 14.—Frequency Distribution of Vehicle Speeds on 3-Lane Highways.

3-LANE HIGHWAY MOST EFFICIENT WHEN TRAFFIC IS EVENLY DIVIDED BY DIRECTION

A similar analysis was made of traffic on the same 3-lane highways during periods when traffic in the direction of heaviest travel did not exceed 60 percent of the total and averaged between 54 and 57.6 percent at the different traffic densities (table 3). For this condition, speeds for corresponding total traffic volumes were a little higher than when about two-thirds traveled in one direction. The most significant difference occurred at the highest comparable traffic densities of about 1,500 vehicles per hour when there was a difference of 1.9 miles per hour in the average speed and a difference of 2.6 miles per hour in the speed of the slowest-moving lane of traffic. The total number of vehicles making use of the center lane was about the same in both cases.

Table 3.—*Distribution of vehicles between lanes and average speeds on 3-lane highway during various traffic densities while about half of all traffic traveled in each direction*

	310 vehicles per hour			670 vehicles per hour			1,248 vehicles per hour			1,530 vehicles per hour		
	Vehicles		Average speed	Vehicles		Average speed	Vehicles		Average speed	Vehicles		Average speed
	Number per hour	Percent		Number per hour	Percent		Number per hour	Percent		Number per hour	Percent	
Traffic in direction of lesser density:												
Right lane	129	41.6	36.6	290	43.3	38.2	504	40.4	37.3	600	39.2	38.6
Center lane	11	3.6	44.3	18	2.7	45.7	45	3.6	46.1	49	3.2	49.0
Total	140	45.2	37.2	308	46.0	38.6	549	44.0	38.0	649	42.4	39.4
Traffic in direction of greater density:												
Right lane	158	51.0	37.7	346	51.6	39.9	599	48.0	36.1	696	45.5	37.6
Center lane	12	3.8	44.7	16	2.4	45.9	100	8.0	44.5	185	12.1	43.8
Total	170	54.8	38.2	362	54.0	40.2	699	56.0	37.3	881	57.6	38.9
All traffic:												
Center lane	23	7.4	44.5	34	5.1	45.8	145	11.6	45.0	1,296	15.3	44.9
Right lanes	287	92.6	37.2	636	94.9	39.1	1,103	88.4	36.6	234	84.7	38.1
Total	310	100.0	37.8	670	100.0	39.5	1,248	100.0	37.6	1,530	100.0	39.1

The conclusions reached from both analyses for 3-lane level tangent sections of highway carrying few trucks were:

1. At any one point on a 3-lane highway relatively few vehicles are traveling in the center lane. The maximum number that can be in the center lane is about 300 per hour, regardless of the total traffic volume, when up to 70 percent of the total traffic is traveling in one direction.
2. Although there is a very marked drop in the average speed of traffic in the outside lanes with an increase in volume, there is no drop in the speeds of vehicles in the center lane.
3. As long as the hourly traffic volume traveling in one direction does not exceed 70 percent of the total traffic, the center lane will be used by vehicles traveling in both directions.
4. The average speed of all vehicles was slightly higher when the traffic was nearly evenly divided by direction than when two-thirds traveled in one direction.

Other obvious facts are:

1. At places where the sight distance is restricted, the use of the center lane for passing is dangerous, so in effect a 3-lane highway will carry only two lanes of traffic at such points.
2. A 3-lane highway having even one restricted sight distance cannot carry more vehicles per hour in one direction than the number that can crowd into one traffic lane.

It may be concluded, therefore, that a 3-lane highway is more efficient when traffic is evenly divided by directions than when two-thirds of the traffic travels in one direction. There is a possibility that a two-directional, 3-lane highway would be more efficient when practically all traffic moved in one direction than when traffic was evenly divided, but the percentage traveling in one direction in such a case would be closer to 100 than to 67. However, since no data were available for this condition, this possibility could not be explored.

DESIRED NUMBER OF PASSINGS PER MILE OF HIGHWAY CALCULATED

By knowing the frequency distribution of vehicle speeds for a particular hourly traffic volume on a tangent section of highway, it is possible to calculate the approximate number of passings that will take place per hour per mile of highway. As an example, if there were 10 drivers traveling 50 miles per hour and 20 drivers traveling 40 miles per hour, each hour in the same direction on a highway, each driver traveling 50 miles per hour would, on an average, pass a vehicle traveling 40 miles per hour every 12 minutes. The total number of passings per hour on each mile of highway would be:

$$P=\frac{AB\ (F-S)}{FS}$$

where

P=number of passings per hour on each mile of highway,
A=number of the faster moving vehicles per hour,
B=number of the slower moving vehicles per hour,
F=speed of the faster moving vehicles in miles per hour,
S=speed of the slower moving vehicles in miles per hour,

and in the above example

$$P=\frac{10\times 20\ (50-40)}{50\times 40}=1$$ passing per mile of highway, per hour.

In a similar manner the number of passings per hour on each mile of highway for any frequency distribution of speeds can be calculated by obtaining a summation of the number of passings made by vehicles moving at each speed passing vehicles moving at each lower speed.

When the number of passings as calculated in this manner from the frequency distribution of speeds recorded during a large number of studies on tangent sections of highway are compared with the passings that actually took place during the same period, the calculated number is generally somewhat higher than the actual number. The difference is no doubt caused by the fact that the speed frequencies represent speeds at a particular point on the highway rather than the distribution of average speeds for the vehicles over the entire portion of the highway having the same design features as the study location. From a frequency distribution showing 10 percent of the vehicles traveling over 50 miles per hour, it is not correct to assume that 10 percent of the drivers traveled over 50 miles per hour all the time under similar conditions. Neither is it correct to assume that all drivers traveled over 50 miles per hour 10 percent of the time. However, the first assumption is more nearly correct than the second for the short time period required by each vehicle to traverse the 0.2 mile section of highway on which the number of passings was recorded. A passing as recorded for this study consisted of two vehicles changing position with respect to one another between the time they entered and left the study section. Whether or not the entire passing maneuver was completed within the study section was not considered.

If it were not for the interference between vehicles as the traffic volume on a highway increases, all drivers could travel at their desired speeds regardless of the traffic density and the distribution of speeds would be the same as the free-speed distributions. In such a case, the total number of passings per hour made by vehicles traveling in one direction on the highway would increase as the square of the hourly traffic volume in that direction.

Using the free-speed distributions for each 2- and 3-lane highway section, the numbers of passings per mile of highway per hour were calculated for the different hourly traffic volumes at each location and compared with the numbers of passings that occurred. Figure 15 shows the percentages that the actual number of passings performed were of the desired number of passings, or passings that would have been performed providing the drivers could have traveled at their desired speeds. Individual points are shown only for the 2-lane highways, each point representing the result for a particular traffic volume at one of 12 level tangent locations.

Although there is a scattering of points at all hourly traffic volumes, the straight-line relationship calculated by the method of least squares shows a very definite decrease in the percentage with an increase in the total traffic density. The line for 3-lane highways was based on data for the same locations as those for curve E of figure 8. Data for both highway types include a total of 11,005 passings performed during the time that 91,866 vehicles traversed the 16 study sections each 0.2 mile long.

It is significant that the hourly traffic volumes of 1,933 vehicles and 3,490 vehicles when no passings could be made on the 2- and 3-lane highways, respec-

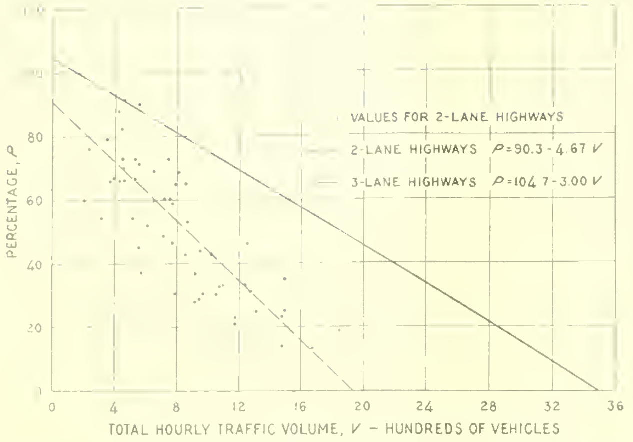

Figure 15.—Percentage of Desired Number of Passings That Are Performed on Level Tangent 2- and 3-Lane Highways.

tively, are approximately the same as the possible capacities of the corresponding highways determined by using the mean difference in speed between successive vehicles as an index of the passing possibilities.

Figure 16 shows the actual number of passings compared with the required number of passings for drivers to maintain their free speeds on a 2-lane highway with two-thirds of the traffic in one direction where the variation in speed was great enough to cause a total of 18.5 passings per mile per hour at a volume of 100 vehicles per hour in one direction. This figure represents a typical 2-lane level tangent highway carrying few trucks and with a frequency distribution of free speeds as shown by the curve for 2-lane highways in figure 5.

The total number of passings required for all drivers to maintain their desired speed increases as the square of the traffic volume. Actually, however, the total number of passings that occur increases with an increase in the total traffic volume up to 1,300 vehicles per hour and then decreases rapidly. To maintain his free speed each driver should be able to increase the number of passings he makes directly as the traffic volume increases. Actually, however, the number of passings made by the average driver increases as the density increases up to 800 vehicles per hour, remains about the same between 800 and 1,200 vehicles per hour, and then decreases with a further increase in the traffic density. The fact that the average driver on a 2-lane tangent highway should increase the number of passings he makes as the traffic volume goes above 800 vehicles per hour, but can make no material increase due to the traffic density, is a very important consideration in the determination of practical capacities for rural highways.

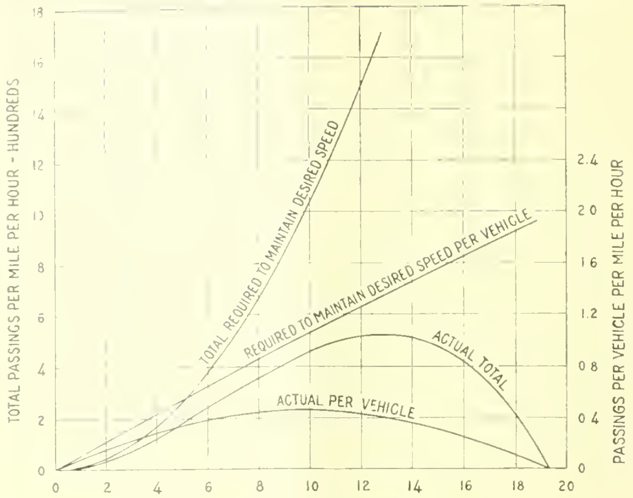

Figure 16.—Comparison of Actual Number of Passings Performed and Number That Would Be Required at Various Traffic Volumes for All Vehicles to Maintain Their Free Speed on 2-Lane Highway With Two-thirds of Total Traffic in One Direction.

VEHICLE SPEEDS IN 1934 AND 1939 COMPARED

Another important factor in the determination of practical capacities, especially for design purposes, is the speeds at which drivers travel at the present time and the speeds at which they will desire to travel at some future date. Until recent years little data have been available regarding speeds on rural highways, but it is known that motor-vehicle speeds increased rapidly until about 1930. During recent years the performance of motor vehicles has continued to improve and there has been a marked improvement in rural highways. Both have tended to increase the average travel speed on rural highways. However, estimates of future performance must be based on speed trends during recent years on modern, well-designed highways. The capacity studies in 1939 included one location of this type that had also been included in the 1934 studies, a section of 3-lane highway. From 1934 to 1939, the paved surface width, alinement, and other general features had not been changed. The speeds of over 8,000 vehicles were obtained during the studies in each of the years. Weather conditions, hours of study, etc. were also the same.

Table 4 shows a comparison of the average speeds for the 2 years. It is rather surprising to find that the average speed for the free-moving vehicles, that is, those that were traveling without interference from other vehicles, or at the driver's desired speed, was slightly lower in 1939 than in 1934. However, at the highest traffic density recorded, the speeds during 1939 were higher than in 1934. In other words, an increased traffic volume had a greater effect on the average speed in 1934 than in 1939.

Table 4.—*Comparison of speeds during 1934 and 1939 on the same 3-lane highway during similar traffic densities*

1934 studies		1939 studies	
Traffic density	Average speed	Traffic density	Average speed
Vehicles per hour	*Miles per hour*	*Vehicles per hour*	*Miles per hour*
(1)	40.8	(1)	39.6
1,069	36.8	1,072	35.7
1,391	36.6	1,338	37.0
1,544	34.4	1,517	37.2
		1,684	35.3

[1] Free-moving vehicles.

Table 5 shows that in 1939 there was a greater variation in speeds than in 1934. In 1934 only 0.5 percent of the free-moving vehicles traveled less than 25 miles per hour and 0.6 percent traveled over 60 miles per hour, while in 1939 5.2 percent traveled less than 25 miles per hour and 3.4 percent traveled over 60 miles per hour.

There was a corresponding or even greater difference between the two years for the extreme speeds at the heaviest comparable densities studied. Since trucks were a negligible portion of the total vehicles on this road during both years, they do not account for this difference. However, a much longer road section was used to determine the speeds for 1934 than for 1939. With the shorter section there was a greater likelihood of obtaining extreme speeds and therefore little significance can be placed on the fact that the results show a greater variation in speeds for the 1939 than the 1934 values.

The important conclusion that can be drawn from this comparison is that there was a relatively small change in speeds from 1934 to 1939 under similar highway and traffic density conditions. This is very significant since the comparison has been made on a highway where there should be the greatest probability of increased speed due to improved vehicle performance. The drivers utilized the potential speeds of their vehicles to a greater extent in 1934 than in 1939.

TABLE 5.—*Comparison of distribution of speeds during light and heavy traffic volumes on same highway in 1934 and 1939*

Speed group (miles per hour)	Free-moving vehicles		Heavy traffic density (about 1,500 vehicles per hour)	
	1934	1939	1934	1939
	Percent	*Percent*	*Percent*	*Percent*
Below 20	0.1	2.7	0.3	2.2
20–24	.4	2.5	1.5	3.2
25–29	1.9	3.9	10.0	5.9
30–34	12.0	23.4	33.6	33.1
35–39	31.8	17.7	29.8	21.7
40–44	25.4	26.1	16.7	18.7
45–49	17.1	12.9	6.0	8.1
50–54	9.1	3.9	1.7	2.6
55–59	1.6	3.5	.2	2.3
60–up	.6	3.4	.2	2.2
Total	100.0	100.0	100.0	100.0

At present it is not possible to predict with any degree of accuracy what the maximum highway speeds in the future will be. The development of the airplane and its construction by mass production methods will undoubtedly tend to reduce the demand for exceedingly high-speed operation of the motor vehicle. Based on recent trends, there will be an increase in speed chiefly by the modernization of substandard highways and by a reduction in the percentage of drivers who travel at exceedingly low speeds on highways designed for high-speed operation, but there is no basis for the assumption that maximum motor-vehicle speeds will be materially higher than they have been on our modern, well-designed highways. Apparently, maximum speeds of passenger cars have been and will continue to be governed to a larger extent by characteristics of the driver, such as his reaction time and ability to see other vehicles and estimate their speeds, than by the possible speed of the vehicle he is operating.

CHARACTERISTICS OF TRAFFIC FLOW USEFUL IN DETERMINING PRACTICAL CAPACITIES GIVEN

When the preliminary results of these capacity studies were published in 1939, a number of figures were presented showing certain characteristics of traffic flow on tangent highway sections that have since been very useful in the determination of practical capacities for different highway conditions. The results as given by these figures have been verified and their possible application expanded by checking them with the results of additional and more recent data. One of the figures presented [1] showed that on 2-lane tangent highways operators of vehicles at or above a time spacing of 9 seconds from the preceding vehicles are not influenced by the speed of the preceding vehicle, but drivers of vehicles spaced at less than 9 seconds govern their speeds to an increasingly greater extent by the speed of the preceding vehicles as the spacing decreases.

The numbers of vehicles traveling at various time spacings from a preceding vehicle on a typical 2-lane tangent highway are shown by figure 17. Similar data representing other 2- and 3-lane tangent highways show that practically the same distribution of time spaces occurs for the same traffic volume in one direction regardless of the average vehicle speed, distribution of speeds, or whether or not the vehicles appeared to be bunched. Any marked difference between the distribution on the various highways generally occurred in the number of vehicles at the extremely short time spacings.

Likewise, the time spacings between successive vehicles traveling in one direction on 4-lane highways regardless of the traffic lane the vehicles were in are shown by figure 18. There was a more uniform distribution of the shorter time spacings and fewer of the extremely long time spacings on the 4-lane roads than on the 2-lane roads for corresponding hourly traffic volumes.

Were it not for the wide variation in vehicle speeds and resulting variation in the time spacings between vehicles, the determination of practical highway capacities and numerous other traffic problems would be relatively easy. In this connection it is rather surprising, when reviewing published material regarding the movement and delay of vehicles under certain conditions, that many theoretical derivations are based on the assumption that all time spaces are equal. In most cases entirely different results would be obtained if consideration were given to the variation that actually exists. The relation between the average time spacing for a given traffic volume and the spaces that exist on 2- and 4-lane tangent rural highways is given by figures 19 and 20. At a traffic volume of 600 vehicles per hour in one direction on a 2-lane highway, instead of a vehicle passing a given point on the highway every 6 seconds, 58 percent of the vehicles will pass within 3 seconds (half the average interval) of another vehicle and there will be an appreciable number of times during the hour that a time interval exceeding 24 seconds will elapse between two vehicles.

For the solution of certain traffic problems involving time spacings between vehicles, such as calculations regarding the average wait before a space sufficiently long occurs in a stream of traffic to permit crossing it with safety, or determinations of the percentage of time that the lane normally used by oncoming traffic may be used to perform passing maneuvers, it is necessary to know the percentage of the total time that the spaces between vehicles are in excess of certain time values in addition to the number of times per hour a space exceeding a certain value occurs. Figures 21 and 22 have been constructed to show this information for tangent sections on 2- and 4-lane rural highways where drivers are not influenced by traffic lights at intersections. An analysis of data for a number of

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[1] Preliminary results of Highway Capacity Studies, by O. K. Norman. PUBLIC ROADS, February 1939, p. 228.

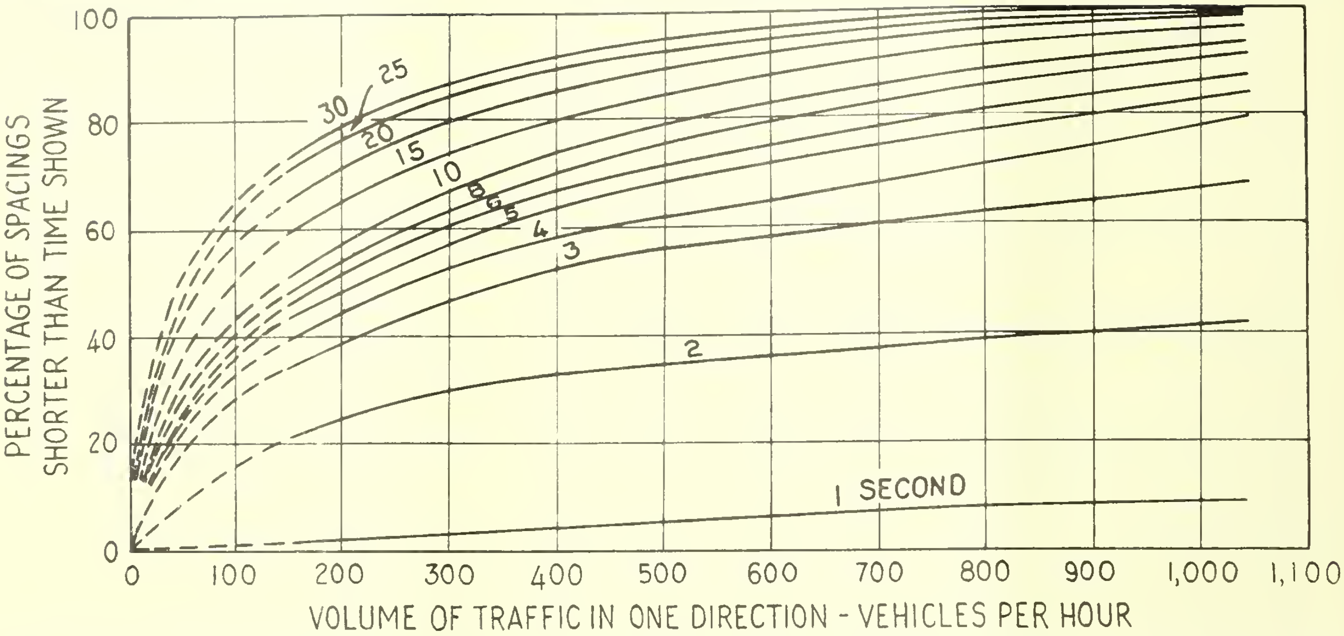

Figure 17.—Frequency Distribution of Time Spacings Between Successive Vehicles Traveling in the Same Direction at Various Traffic Densities on Typical 2-Lane Rural Highway.

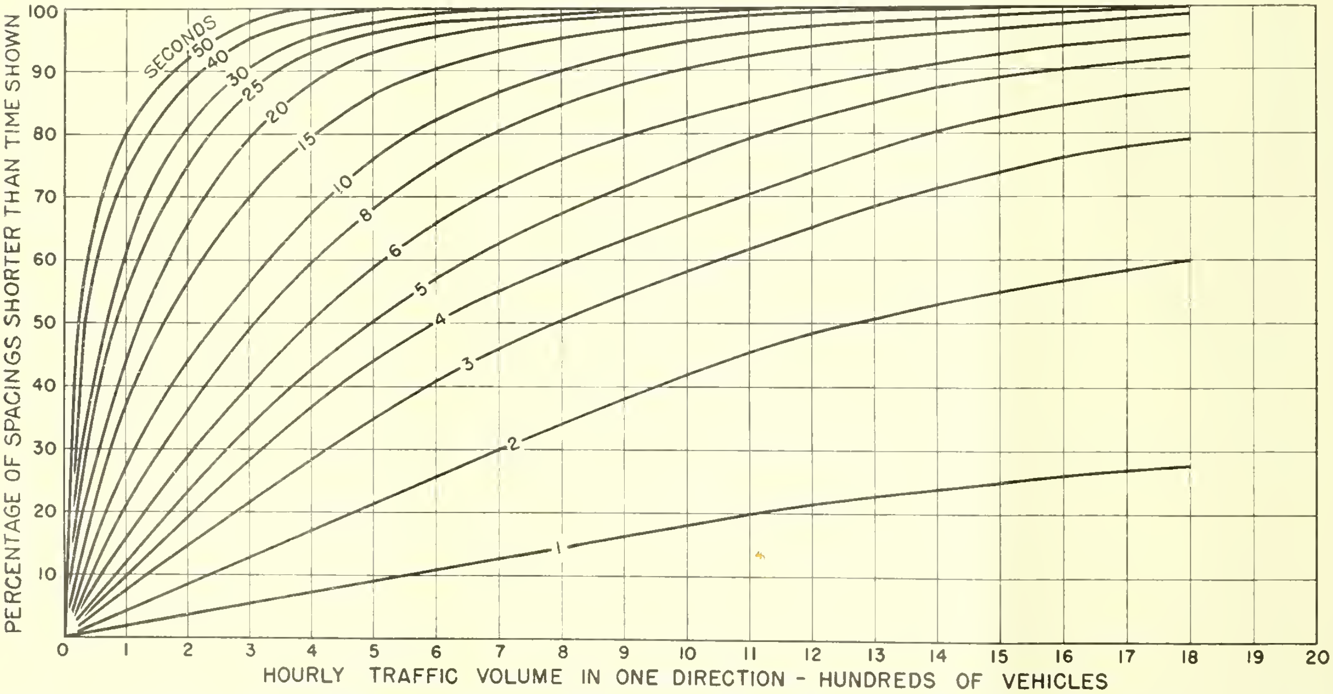

Figure 18.—Frequency Distribution of Time Spacings Between Successive Vehicles Traveling in the Same Direction at Various Traffic Densities on Typical 4-Lane Rural Highway.

locations showed that there was some variation in corresponding values at different locations, but for all practical purposes the values as given by figures 21 and 22 are sufficiently accurate for all 2- and 4-lane highways, respectively. Corresponding values for 3-lane highways were not consistent, generally being in the neighborhood of those for 2-lane highways but oftentimes approaching those for 4-lane highways.

In combination with figures 17 and 18, figures 21 and 22 may be used to solve a number of traffic problems for which it is either impossible or impractical to obtain more accurate results by actual field studies.

PRACTICAL APPLICATION OF RESULTS ILLUSTRATED

To illustrate one of the many uses of figures 21 and 22, assume it is desired to compare the average time lost by a driver in safely crossing 2- and 4-lane highways. Assume that a driver on a minor east and west road approached a 2-lane arterial highway carrying a total of 1,200 vehicles per hour with 400 northbound and 800 southbound and that it will require 5 seconds to cross and be clear of traffic on the through route. Figure 17 shows that 64 percent of the spacings at a volume of 400 vehicles per hour are equal to or less

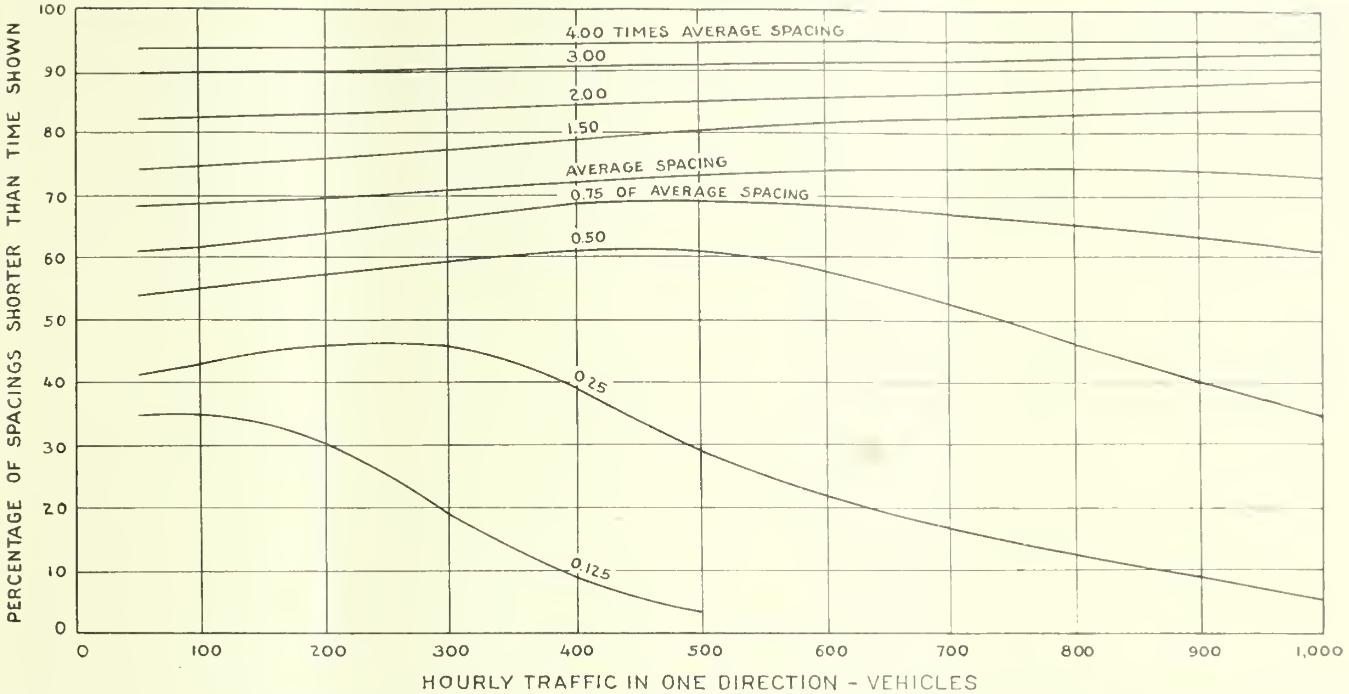

Figure 19.—Cumulative Percentage of Vehicles Traveling at Time Spacing Shorter Than Multiples of Average Spacing for Various Hourly Traffic Volumes on Typical 2-Lane Rural Highway.

than 5 seconds, therefore spaces greater than 5 seconds occur 144 times per hour in the northbound traffic. Likewise, spaces greater than 5 seconds occur 180 times per hour in the southbound traffic. The spaces in the northbound traffic are in excess of 5 seconds 66.5 percent of the time and those in the southbound traffic are in excess of 5 seconds 40 percent of the time (fig. 21). So, out of 100 drivers on the minor road, 27 drivers $(0.40\times0.665\times100)$ will be able to cross without any delay after coming to a stop. The other 73 will be delayed, 20 by traffic in both lanes $[100\ (1.00-0.40)\ (1.00-0.665)]$, 19 by traffic in the northbound lane $\left(53\times\frac{0.335}{0.335+0.600}\right)$, and 34 by traffic in the southbound lane $\left(53\times\frac{0.600}{0.335+0.600}\right)$. After the 34 delayed by traffic in the southbound lane have waited an average of 6 seconds $\left[\frac{1}{2}\left(\frac{0.60\times3{,}600}{180}\right)\right]$, 23 will be able to cross with no further delay (34×0.665) and the other 11 will not be able to cross because of traffic in the northbound lane. By continuing the calculations it can be shown that the average delay for the 73 delayed vehicles will be 9.5 seconds and the average delay for all 100 drivers will be about 7 seconds. Some drivers will be delayed up to 50 seconds. In a similar manner the average delay to cross 2- or 3-lane highways, or 4-lane divided or undivided highways with any traffic volume on the arterial highway, may be calculated provided the crossing time is known.

Based on a reasonable acceleration value it has been estimated that 5 seconds are required to cross a 2-lane highway, 6 seconds to cross a 4-lane undivided highway, 7 seconds to cross the first 2 lanes of a 4-lane divided highway and come to a stop within a 30-foot median strip, and 5 seconds to cross the second 2 lanes. Using these time periods, the data shown by figures 17, 18, 21, and 22, and assuming that a safe crossing will not be made when it is necessary for vehicles on the arterial highway to slow down to avoid an accident, figure 23 has been constructed. The values shown are applicable only to vehicles on a cross road carrying very little traffic and do not include the delay in stopping for the "stop" sign.

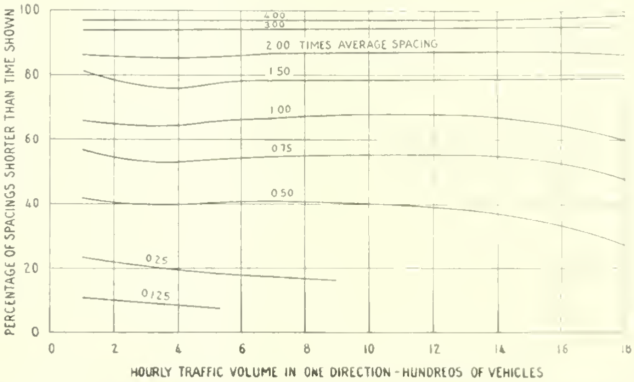

Figure 20. Cumulative Percentage of Vehicles Traveling at Time Spacings Shorter Than Multiples of Average Spacing for Various Hourly Traffic Volumes on Typical 4-Lane Rural Highway.

The average delay is lower at the 2-lane highway than at the 4-lane undivided highway for the same hourly volume not only because a slightly shorter time is required to cross the 2-lane highway, but also because traffic cannot travel as freely on the 2-lane highway and becomes more bunched, resulting in longer open spaces that permit crossings to be performed. Up to a total of 1,400 vehicles per hour, the delay is about the same when crossing 2-lane and 4-lane divided highways. Above 1,400 vehicles per hour it is easier to cross the divided highway and considerably less delay will be experienced by drivers that start the maneuver by

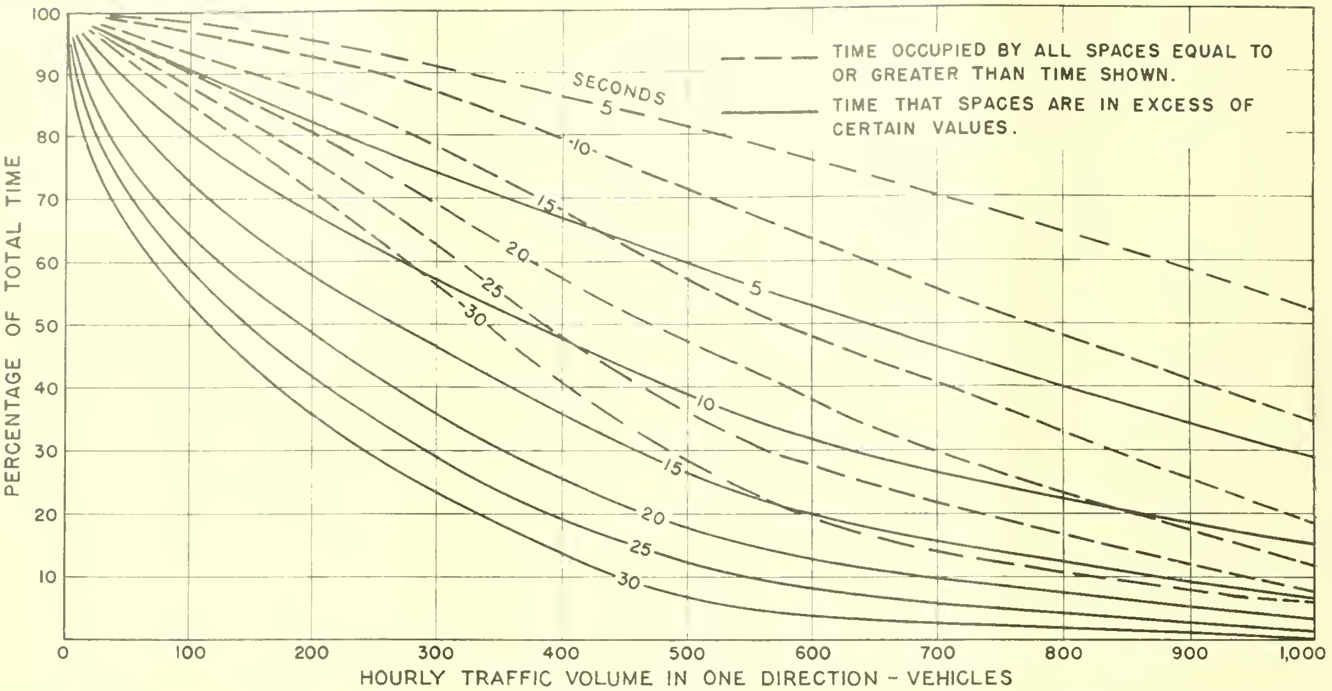

Figure 21.—Percentage of Total Time Occupied by Various Time Spacings Between Vehicles Traveling in the Same Direction and Percentage of Total Time That Spacings Are in Excess of Certain Values on Typical 2-Lane Rural Highway.

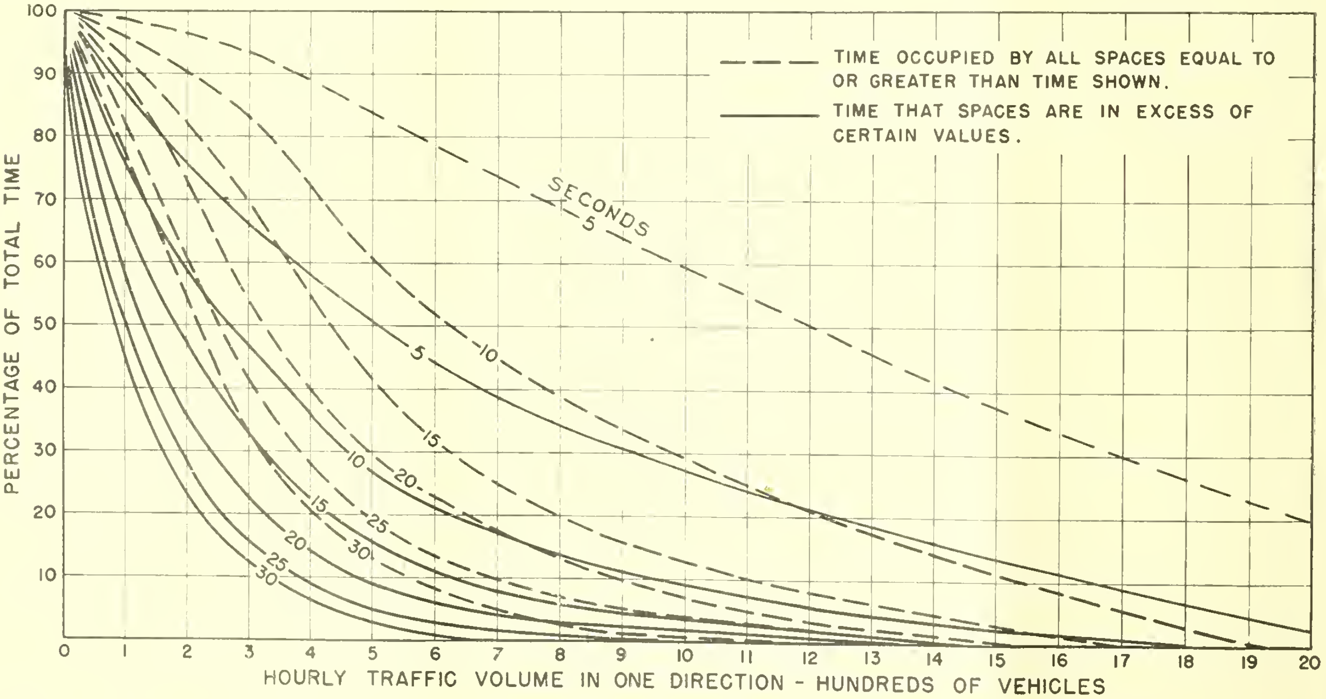

Figure 22.—Percentage of Total Time Occupied by Various Time Spacings Between Vehicles Traveling in the Same Direction and Percentage of Total Time That Spacings Are in Excess of Certain Values on Typical 4-Lane Rural Highway.

crossing the two lanes carrying the lighter traffic volumes than by drivers that first cross the lanes carrying the heavier traffic volumes.

Using the same basic data, calculations may be made to determine need for the installation of traffic signals at rural intersections with greater accuracy than would otherwise be possible. However, the information shown in figures 17 and 21 is most useful in a study of practical capacities for 2-lane roads by supplying data of value for an analysis of the extent to which oncoming traffic restricts the use of the left lane for passing purposes. To illustrate, assume that a driver is trailing a slow-moving vehicle and that to pass this vehicle he must encroach on or be in the left lane for a period of 10 seconds, traveling at an average speed of 30 miles per hour. The "hole" or time space between successive

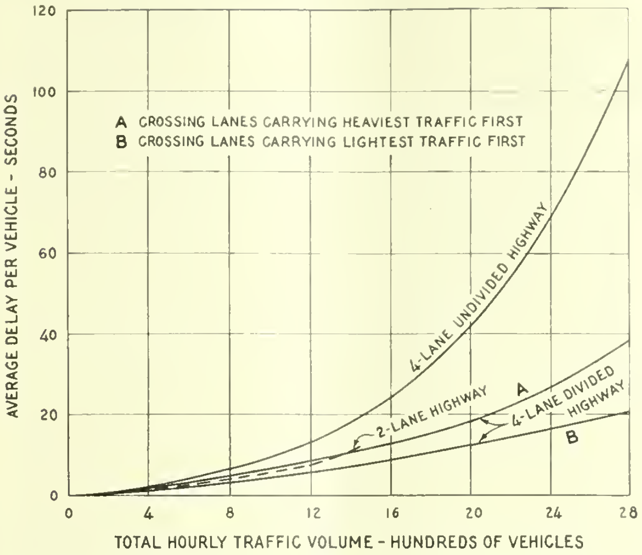

Figure 23.—Delay for Individual Vehicles to Cross 2- and 4-Lane Highways Carrying Two-thirds of Total Traffic in One Direction.

vehicles in the opposing traffic lane could not be less than 20 seconds $\left(10\times\frac{30+30}{30}\right)$ if the oncoming vehicle met immediately after completing the maneuver is traveling at 30 miles per hour, not less than 25 seconds $\left(10\times\frac{20+30}{20}\right)$ with the oncoming vehicle traveling at 20 miles per hour, and not less than 16 seconds $\left(10\times\frac{50+30}{50}\right)$ with the oncoming vehicle traveling at 50 miles per hour.

PASSING POSSIBILITIES A FUNCTION OF TRAFFIC DENSITY

Likewise, to perform a passing maneuver requiring 10 seconds in the left lane at a speed of 50 miles per hour with the oncoming vehicle traveling 20, 30, or 50 miles per hour would require time spacings between vehicles in the opposing traffic lane of not less than 35, 27, or 20 seconds, respectively. How often the required time spaces will occur with a corresponding speed for the approaching vehicle is not shown by figure 21. However, analysis of the detailed data shows that the sum of the opportunities to pass at the different time spacings with oncoming vehicles traveling at different speeds may be determined from figure 21 by use of the following equation:

$$T=\frac{T_p(S_0+S)}{S_0}$$

where

T = equivalent spacing in seconds to be used in determining opportunities to pass from figure 21,

T_p = time required in left lane to complete passing maneuver,

S_0 = average speed of opposing traffic in left lane, and

S = average speed of passing vehicle while in the left lane.

For example, assume that it is desired to find the average time a driver wanting to pass a vehicle traveling 20 miles per hour on a 2-lane tangent section of highway carrying 400 vehicles per hour in the opposing direction will be required to follow the slow vehicle before an opportunity to pass occurs. Assume also that 10 seconds will be required to complete the maneuver at an average speed of 30 miles per hour, and that the average speed of oncoming traffic is 40 miles per hour; then $T=\frac{10\ (40+30)}{40}=17.5$ seconds.

A spacing of 17.5 seconds or greater will occur 18 times (fig. 17) for every 100 cars in the opposing lane and 32 percent of the time (at volume of 400 in fig. 21) it will be possible to start the passing. So, for every 100 times a driver overtakes a vehicle traveling 20 miles per hour, it will be possible for him to start the passing maneuver with no delay 32 times and the other 68 times he will be required to follow the slow vehicle for some distance before he has an opportunity to pass. The average time in seconds that the slow vehicles must be trailed whenever the passing cannot be started immediately will be

$$F=\tfrac{1}{2}\left[\frac{3600(1.00-P_E)}{P_N V_0\left(\frac{S_0+S_p}{S_0}\right)}\right]$$

where

F = average time in seconds slow vehicle must be followed before opportunity to pass occurs,

P_E = ratio of time that spacings in opposing lane are in excess of T seconds for the particular traffic volume in the opposing lane to total time,

P_N = ratio of number of spaces in the opposing lane that are in excess of T seconds to total number of spaces,

V_0 = hourly traffic volume in opposing lane,

S_0 = average speed in miles per hour of traffic in opposing lane, and

S_p = speed in miles per hour of vehicle to be passed.

Then

$$F=\tfrac{1}{2}\left[\frac{3600(1.00-0.32)}{0.18\times 400\left(\frac{40+20}{40}\right)}\right]=11.3 \text{ seconds.}$$

The average distance that it will be necessary to trail the slow vehicle moving 20 miles per hour before an opportunity to pass occurs will be 332 feet.

Figure 24 shows similar delay values for a number of different conditions using the average speeds of oncoming traffic as shown for 2-lane highways by figure 6. The two assumed values, 40 and 25 miles per hour, for the speed of the vehicle to be passed are the maximum and minimum speeds of vehicles that other vehicles should be able to pass, without an exceedingly long delay, if a 2-lane level highway is not to be considered congested. The speed of the passing vehicle while in the left lane is based on the preliminary results of the passing studies [5] which show that, on an average and in the majority of cases when a passing maneuver is delayed, the passing vehicle travels about 10 miles per hour faster than the passed vehicle during the maneuver. The two time values given for each condition as the period that the passing vehicle is in the left lane cover the range as shown by the passing studies for the majority of maneuvers performed under the two conditions.

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[5] Passing practice on rural highways, by C. W. Prisk. Highway Research Board Proceedings, 1941.

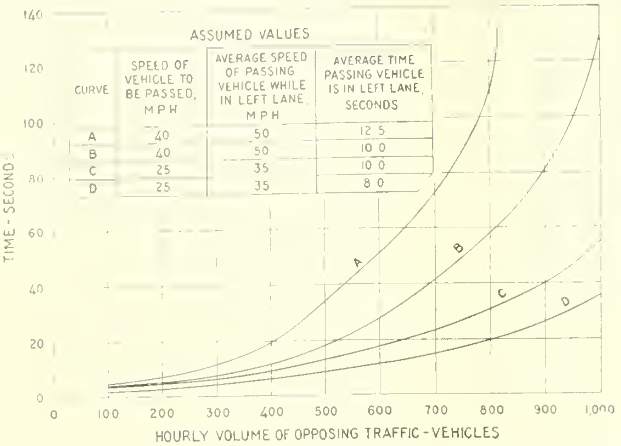

Figure 21.—Average Time in Seconds That Driver Desiring to Pass Will Be Required to Trail a Slow-moving Vehicle Before an Opportunity to Pass Will Occur in the Opposing Traffic Lane Under Certain Conditions on a 2-Lane Highway.

Curve A (fig. 24) shows that opportunities to pass a vehicle traveling 40 miles per hour and requiring 12.5 seconds in the left lane will occur until the density in the opposing traffic lane reaches about 800 vehicles per hour. Likewise, for the condition shown by curve B (fig. 24) an opportunity to pass will occur until the traffic volume in the opposing lane reaches 1,000 vehicles per hour. Curves C and D show that opportunities to pass vehicles traveling 25 miles per hour occur when the opposing traffic volume exceeds 1,000 vehicles per hour. However, since the openings in the opposing traffic that were used to calculate the passing possibilities are minimum requirements, the drivers performing the passing maneuvers must be able to judge the speeds of the oncoming vehicles to take full advantage of their passing opportunities and the return to the right-hand lane must be made before meeting the oncoming vehicle.

The passing studies show that drivers do not take full advantage of their passing opportunities even under congested conditions, but often allow a considerable portion of the time space in the opposing traffic to pass before starting the maneuver. Also the majority of drivers will not start the maneuver unless the space between vehicles in the oncoming lane is considerably greater than the minimum requirement. This results in few passings being performed when the volume of opposing traffic exceeds 1,000 vehicles per hour and, therefore, curve B probably represents fairly accurately the actual trailing times before the slowest vehicles are passed.

HIGHEST PRACTICAL CAPACITY OF A RURAL 2-LANE HIGHWAY IS 800 VEHICLES PER HOUR

Based on the time that the slow-moving cars must be trailed before a passing maneuver can be made, a traffic volume of 400 vehicles per hour in each direction is the highest density that may be considered as the practical capacity for a 2-lane level tangent section of highway. Regardless of whether the actual average trailing time is as shown by curve B or a longer period as shown by curve A (fig. 24), 400 vehicles per hour is the highest volume in the opposing lane that will not cause a relatively large increase in the trailing time as compared to the amount that the traffic volume is increased. From curve B, increasing the opposing traffic volume from 300 to 400 vehicles per hour results in an increase of 33 percent in the number of vehicles and 40 percent in the trailing time, while increasing the volume from 400 to 500 vehicles per hour results in a 25 percent increase in the number of vehicles and a 61 percent increase in the trailing time. Using curve A there is an increase of 64 percent in the trailing time when the opposing traffic volume increases from 300 to 400 vehicles per hour and an increase of 85 percent when the hourly volume increases from 400 to 500 vehicles. The ratio between the increase in volume and the increase in delay decreases very rapidly as the opposing traffic volume goes above 400 vehicles per hour.

Other traffic characteristics that indicate a total traffic volume of 800 vehicles per hour to be the highest practical capacity for a 2-lane level tangent rural highway carrying few trucks are:

1. Seventy-two percent of the drivers will be at spacings of less than 9 seconds (fig. 17) from the preceding vehicle and they will be governing the speeds of their vehicles to some extent by the speeds of the preceding vehicles.
2. The average driver will only perform 54 percent of the passing maneuvers that would be required for him to maintain his desired speed (fig. 15).
3. As the total traffic volume increases to 800 vehicles per hour, the average driver increases the number of passings he makes. Above 800 he does not increase the number he makes although to maintain his desired speed there should be a continued increase in the number of passings made (fig. 16).
4. With a total traffic volume of 800 vehicles per hour on a rural highway the average driver is restricted to approximately the same extent as when traveling during low traffic densities within a city on an arterial street without traffic lights but where there is a 30-mile-per-hour speed limit (fig. 8).

Eight hundred vehicles per hour is about 40 percent of the possible capacity of a 2-lane rural highway carrying few trucks. The same percentage of the possible capacities of the 3-lane highways is 1,120 vehicles per hour for the study sections in Massachusetts, and 1,460 vehicles per hour for the study sections in New York. Forty percent of the possible capacity for two lanes of the 4-lane sections in Illinois was 1,680 vehicles per hour or a total of 2,520 vehicles for all four lanes assuming that two-thirds of the traffic will be traveling in the one direction. At these densities, the speed differences as shown by figures 6 and 8 are higher on the 3- and 4-lane highways than on the 2-lane highways.

For speed differences corresponding to the speed difference on the 2-lane roads carrying 800 vehicles per hour the 4-lane Illinois highways could carry 3,150 vehicles per hour and the 3-lane highways in Massachusetts or New York could carry 1,600 vehicles per hour. However, since the speed differences were higher on the 3- and 4-lane roads during low traffic densities than on the 2-lane roads, it seems reasonable to believe that the drivers will expect a greater freedom of movement on 3- and 4-lane roads than on 2-lane roads. Practical capacities for 2-, 3-, and 4-lane roads with good alinements and carrying few trucks are therefore in the neighborhood of 800, 1,400, and 2,800 vehicles per hour respectively. This is a ratio of 1:1.75:3.5.

In case local conditions demand that 400 vehicles per hour be the standard adopted for the practical or working capacity for a 2-lane highway with few limiting sight distances, as may well be the case for express or toll highways, corresponding capacities for 3- and 4-lane roads would be 800 and 1,800 vehicles, respectively. This is a ratio of 1:2:4.5. Likewise, for conditions where the working capacity of a 2-lane highway may be considered to be above 800 vehicles per hour, the ratio for 2-, 3-, and 4-lane highways approaches the same ratio as the possible capacities are to one another.

For certain conditions, such as those illustrated by curves F and G in figure 8 where a speed limit is enforced, the practical capacity of two lanes of a 4-lane highway is 2,450 vehicles per hour. Any lower value would result in only a slight increase in the average speed or freedom of movement for the individual drivers, while a volume above 2,450 per hour would result in a marked reduction in both speed and freedom of movement.

EFFECT OF CURVES AND GRADES ON HIGHWAY CAPACITY STUDIED

There are so many types of curves and grades, and they may be combined with level tangent sections to form so many different highway alinements, that a comprehensive study of their effect on highway capacity involves an unlimited number of conditions. Although additional studies have been made, data for only 5 curves and 10 grades involving a total of 62,289 vehicles have been analyzed.

Curves affect the movement of vehicles on 2- and 3-lane highways mainly by restricting passing maneuvers. Theoretical capacities of all highways are probably affected to some extent by the speed at which the various curves can be negotiated safely and the distance spacing between vehicles allowed by the drivers while on a curve as compared to the spacings allowed on tangent sections. However, since the maximum theoretical capacities occur at relatively low speeds, the difference between the maximum theoretical capacities of a curved section of highway and a tangent section is probably not large.

The possible capacity of a 2-lane road is reached on tangent sections of highway when vehicles cannot pass one another. Therefore, an individual curve will cause no reduction in the possible capacity of a highway since it will impose no further restriction to passing than already exists. However, at lower traffic densities, a curve with a restricted sight distance prevents passing maneuvers that could otherwise be performed safely.

Of the few curves for which the data have been analyzed, the one with the greatest degree of curvature and the shortest sight distance that had no apparent effect on traffic was a 4° curve with a minimum sight distance at one point of about 900 feet. The pavement was 20 feet wide and had a superelevation of 0.06 foot per foot. There were only slightly fewer passings performed on the curve than on an equal length of tangent section under similar traffic densities. As a comparison, during low traffic densities, an 11° curve, with a minimum sight distance of 400 feet and superelevated 0.085 foot per foot, caused passenger cars to reduce their average speeds from 43 to 35 miles per hour, busses from 48 to 35 miles per hour, trucks from 37 to 32 miles per hour, and tractor-truck semitrailers from 33 to 30 miles per hour.

In addition to limiting the sight distance along the highway, steep grades also limit the speed at which vehicles, especially heavily loaded trucks, can travel. Table 6 shows the average speed of free-moving trucks, busses, and passenger cars while traveling on 3-, 5-, and 7-percent grades that were approximately ¼ mile long and had level tangent approaches. Excluding the vehicles on the level sections, speed data for 1,611 trucks, 133 busses, and 21,036 passenger cars were used to obtain these averages.

Figure 25 shows the distribution of truck and passenger-car speeds on the various grades. When comparing these distributions, one must remember that (1) the speed distributions include all trucks on the highway, both loaded and empty, (2) no vehicle was restricted by any other vehicle, (3) the grades were only ¼ mile long, and (4) the speeds at the bottom of the grades were not necessarily the same as on the level sections since the drivers could see the grades for some distance and may have speeded up before reaching the grade.

Since some of the data included in these results were obtained during 1934 and 1935 when the engine power of both trucks and passenger cars was considerably less than at present, a comparison was made of the speeds on the different grades in 1934 and 1940. The resulting average speeds were practically identical. Evidently, the increased loads hauled by the trucks have compensated for the increase in horsepower, while the drivers of passenger cars utilized a larger percentage of the vehicles' potential power in 1934 than in 1940.

TABLE 6. *Free vehicle speeds on grades*[1]

Grade	Trucks	Busses	Passenger cars
	M. p. h.	*M. p. h.*	*M. p. h.*
Level	36.9	45.9	45.8
Downgrade:			
3 percent	37.6	46.4	46.5
5 percent	38.9	41.4	42.1
7 percent	34.1	37.4	40.2
Upgrade:			
3 percent	34.3	37.0	43.5
5 percent	26.6	29.3	39.5
7 percent	24.6	26.6	34.4

[1] All grades had level tangent approaches and were about ¼ mile long.

Figure 25 shows that a 3-percent grade ¼ mile long had only a slight effect on the free speeds of passenger cars and that the decrease in speed was about the same in going from a 3- to a 5-percent grade as from a 5- to a 7-percent grade. Most passenger cars are capable of maintaining high speeds on 7-percent grades. The decrease is therefore caused primarily by the operation rather than the possible performance of the vehicle.

Trucks traveled nearly as fast up the 3-percent grades as on the level but there was a large reduction in going from a 3- to a 5-percent grade.

From the speed distributions as shown by figure 25 and using methods previously explained, table 7 has been prepared to show the number of passings that would take place on grades ¼ mile long if all drivers could maintain their free speeds and the traffic volume was 100 vehicles per hour. The increase in the number of passings with an increase in the grade is relatively slight until the grade goes above 5 percent regardless of the percentage of trucks. The increase in the variation in passenger car speeds as the grade increases from 5 to 7 percent causes a larger increase in the desired number of passings than does increasing the number of trucks to 20 percent of all traffic. However, when a desired passing cannot be made, the trucks cause a greater delay since they travel at lower speeds

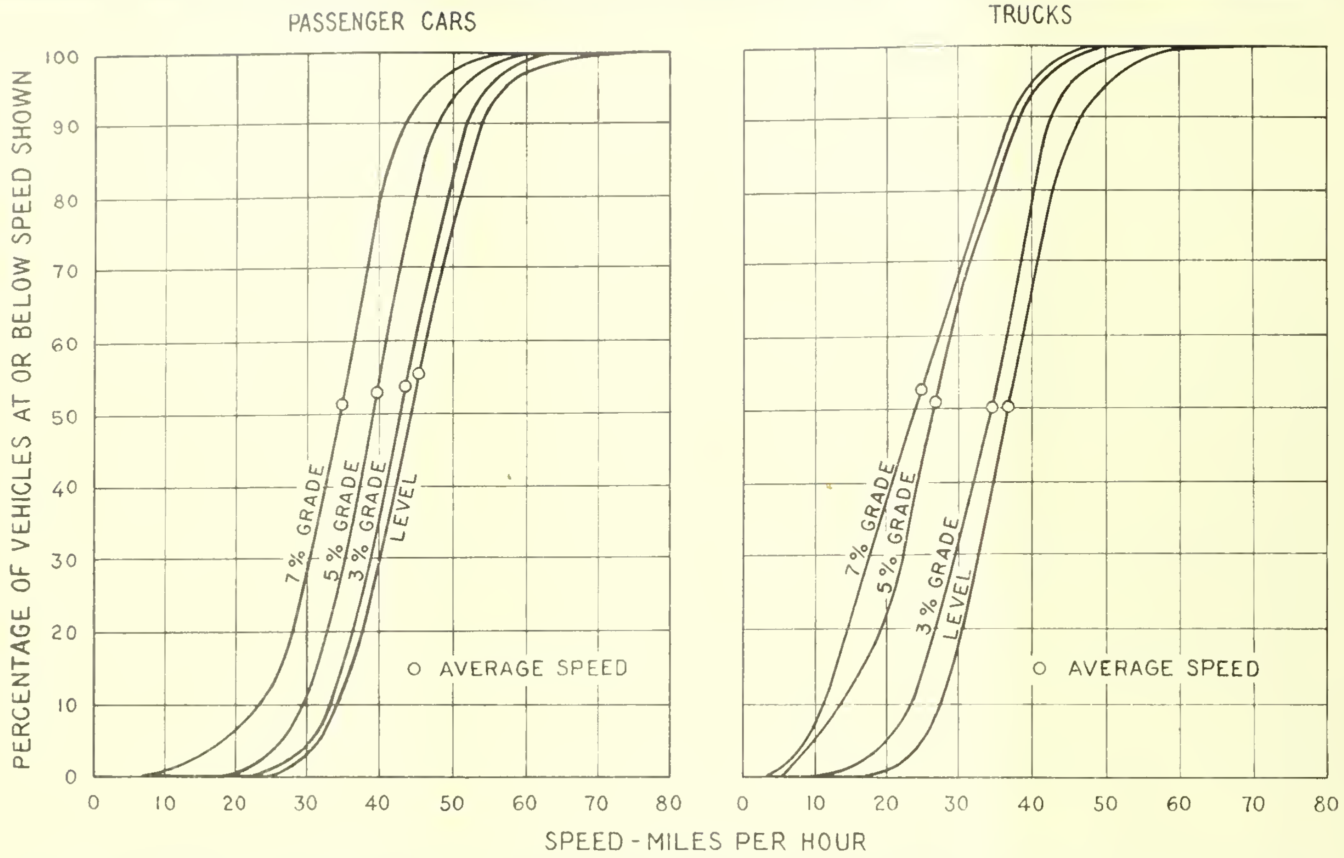

Figure 25. Cumulative Frequency Distribution of Free Speeds up Grades ¼ Mile Long With Level Tangent Approaches.

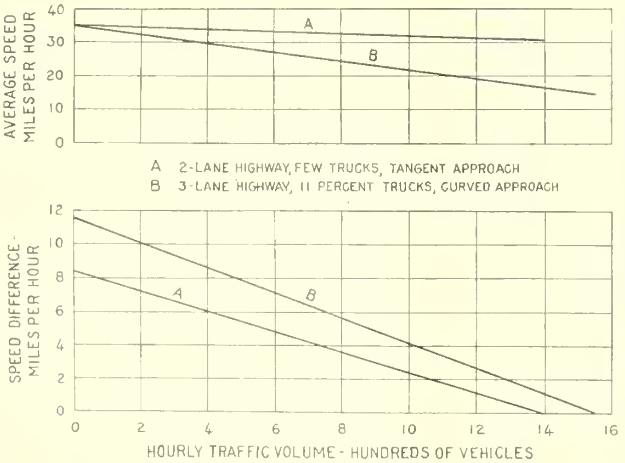

Figure 26.—Average Speeds and Speed Differences on Tangent 7 Percent Grades ¼ Mile Long.

Table 7.—*Desired number of passings per hour to maintain free speeds with 100 vehicles per hour in one direction on ¼-mile section of highway*

Grade, percent	No trucks	Trucks 10 percent of all traffic	Trucks 20 percent of all traffic
	Passings per hour	*Passings per hour*	*Passings per hour*
0	5.6	6.2	7.1
3	5.7	6.4	7.3
[illegible]	6.7	7.1	8.1
[illegible]	12.8	15.5	17.9

From the available data on grades, it has been possible to determine capacity figures for only two highway conditions. One condition represents 7-percent grades ¼-mile long on 2-lane highways that carry few trucks and have level tangent approaches to the grades. The other condition represents 7-percent grades ¼-mile long on 3-lane highways that carry 11 percent trucks and have curved approaches to the grades.

Figure 26 shows that although the average free speed was 35 miles per hour for both conditions, the drop in speed was much greater as the density increased on the 3-lane roads than on the 2-lane roads. Passing was not possible on the grades when the density reached 1,400 and 1,560 vehicles per hour on the 2- and 3-lane highways, respectively. However, in the one case the vehicles averaged 30 miles per hour and in the other they averaged 14.5 miles per hour. The trucks and the curved approaches reduced the possible capacity of the 3-lane highways to a value nearly as low as the value for the 2-lane highways and caused a much larger reduction in speed. Vehicles ceased to pass on the 2-lane highway grades at only seven-tenths of the density at which they stopped passing on level sections.

EFFECT OF SHORT SIGHT DISTANCES ON HIGHWAY CAPACITY EVALUATED

To evaluate the effect that short sight distances at both horizontal and vertical curves have on the practical working capacity of a 2-lane highway, it is necessary to know the number and type of passing maneuvers that the short sight distances prevent from being performed safely. This requires a knowledge of the frequency of the different types of passings under various traffic conditions, the time and space requirements for the different types of maneuvers, and the sight distance or clear space in the opposing traffic lane that must be available before safe drivers will attempt to pass.

While the passing studies on 2-lane highways were designed to provide the main source of information for determining economical sight distance requirements, some very useful information has also been obtained from an analysis of the speed-capacity study data. Figure 27 shows the distribution of speeds for all vehicles and the vehicles that were passed on a 4-lane divided highway during low traffic densities when none of the drivers were prevented from passing other drivers traveling at slower speeds. To illustrate the accuracy of the method previously explained and used to calculate the desired passings from the free-speed distributions, a curve is also shown for the speed distribution of passed vehicles as calculated.

Even though 8 percent of the drivers traveled over 60 miles an hour and a few went over 75 miles an hour, no driver traveling over 60 miles an hour was passed and 95 percent of the passed drivers were traveling less than 50 miles per hour. Since the speeds at this particular location were higher than at any other of the many locations where high speeds were recorded in these studies, the expenditure of additional funds to obtain sight distances greater than those required for a driver to pass a vehicle going 50 miles per hour cannot be justified on any highway, at least not until more concrete evidence is available to show that future speeds will be considerably higher than present speeds. Fifty miles per hour is the 66 percentile value of the free speeds. A more conservative design that would provide for 80 percent of the passings would require a sight distance sufficient to pass a vehicle going 45 miles per hour which is only the 38 percentile value of the free speeds.

Figure 28 shows cumulative speed distribution curves for all vehicles and the vehicles that were passed on a 2-lane tangent section of level rural highway during total hourly traffic volumes up to 1,179 vehicles. At this particular location where drivers were not restricted in any way by sight distances, no vehicle going over 50, 47.5, and 45 miles per hour was passed when the total hourly traffic volumes were 496, 823, and 1,179 vehicles, respectively.

If a portion of this same highway had numerous curves with restrictive sight distances, increasing the sight distances to a value longer than required to pass a vehicle going 45 miles an hour would be of no benefit to the traffic using the highway when the total hourly volume was above 1,179 vehicles per hour, and since the slopes of the curves showing the speed distributions for the passed vehicles change very rapidly between the 90 and 95 percentile values, very little would be gained in increasing the sight distance at any point above the value required to pass a vehicle going 36 miles an hour. Likewise, sight distances longer than those needed to pass vehicles going 39 and 42 miles an hour are of little benefit when the hourly traffic volumes are above 823 and 496 vehicles, respectively. Speeds of 36, 39, and 42 miles per hour are the 14, 30, and 55 percentile speeds, respectively, of the free speed distribution.

The analysis of the passing practice studies has not progressed far enough to show definitely what sight distances drivers require before undertaking to pass vehicles traveling at these speeds, but the average travel distances in the left lane are 576, 621, and 665 feet, respectively.

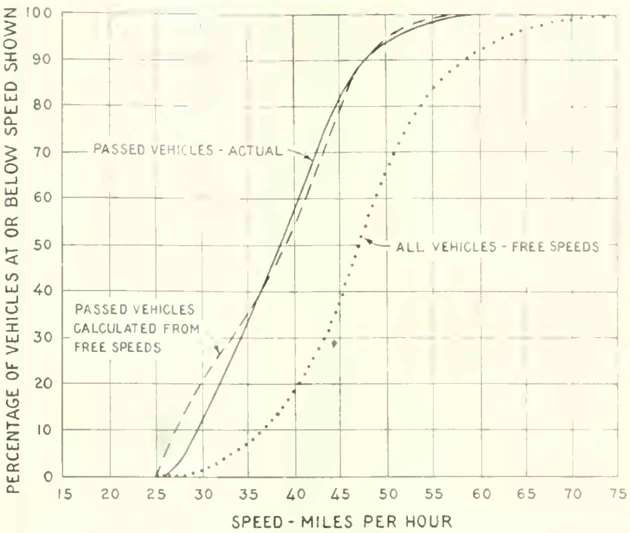

Figure 27.—Cumulative Frequency Distribution of Speeds of All Vehicles and Passed Vehicles on 4-Lane Divided Highway During Low Traffic Densities.

In case satisfactory conditions exist on long tangent sections of this highway when the traffic volume is 800 vehicles per hour, conditions will also be satisfactory on all sections of the highway where there are no sight distances shorter than the one required to pass a vehicle going 40 miles per hour.

The sight distance required to pass a vehicle going at a certain speed will depend on the speed of oncoming traffic, which on a 2-lane highway decreases as the total traffic volume increases. Figure 28 shows that the slope of the cumulative curves for the speed distribution of all vehicles changes very rapidly above the 90 percentile value. The 90 percentile speed value for oncoming traffic when providing adequate passing sight distances seems, therefore, to be the highest that can be justified in obtaining an economical design. As an example, if traffic conditions were satisfactory on the tangent section of highway where data were obtained for figure 28 when there was a total traffic volume of 800 vehicles per hour, any sight distance, on any section of the highway, long enough to permit a driver to pass a vehicle going 40 miles an hour with an oncoming vehicle approaching at a speed of 45 miles an hour cannot be considered a restrictive sight distance. A longer sight distance would increase the freedom of movement at lower volumes but not at the critical volume.

MORE COMPLETE ANALYSIS OF PASSING PRACTICES NEEDED

The relation between the percentage of all vehicles and the percentage of passed vehicles that exceed various speeds at different total hourly traffic volumes, as shown by figure 28, may also be shown as percentile values of the cumulative free-speed distribution. For example, with traffic volumes of 496, 823, and 1,179 vehicles per hour, the fastest vehicles traveled 57½, 54, and 50½ miles per hour, respectively. These speeds are the 99, 97, and 93 percentile values of the free-speed distribution (fig. 28). The 100-percent curve for all vehicles (fig. 29) shows these same percentile values. The other curves on figure 29 show the relation between the cumulative percentile speed values at different traffic volumes and the cumulative percentile speed

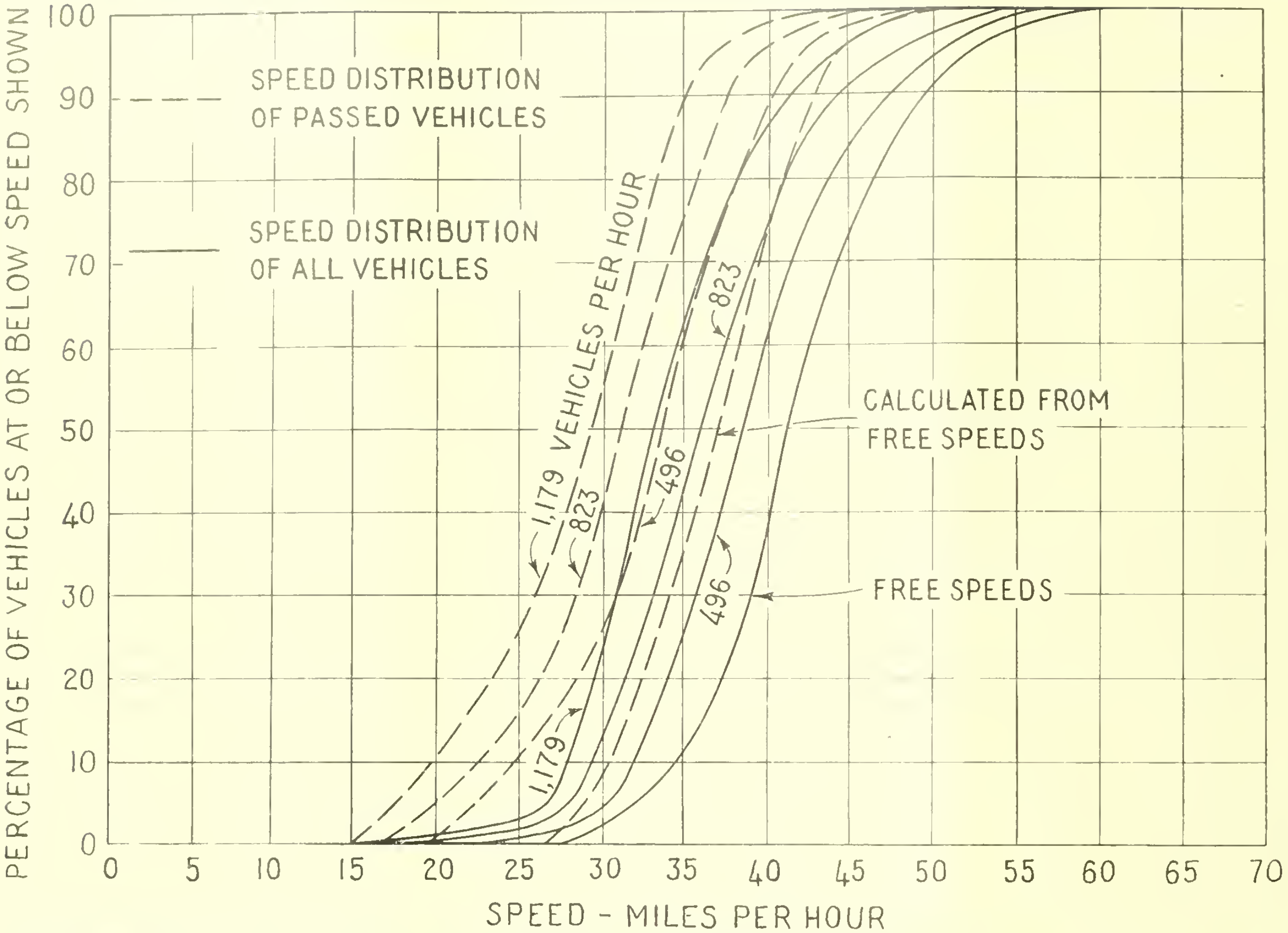

FIGURE 28. CUMULATIVE FREQUENCY DISTRIBUTION OF SPEEDS OF ALL VEHICLES AND PASSED VEHICLES ON 2-LANE TANGENT HIGHWAY DURING VARIOUS TOTAL HOURLY TRAFFIC VOLUMES.

values of the free-moving vehicles. When the total traffic volume was 400 vehicles per hour, the 80 percentile speed for all vehicles was the same as the 70 percentile speed of the free-moving vehicles, the 85 percentile speed for all vehicles was the same as the 77 percentile speed of the free-moving vehicles, etc. Also with 400 vehicles per hour the 80 percentile speed for the vehicles that were passed was the same as the 29 percentile speed of the free-moving vehicles, the 95 percentile speed of the passed vehicles was the same as the 60 percentile speed of the free-moving vehicles, etc.

Similar data for other 2-lane tangent highway sections indicate that the values shown by figure 29 hold true within practical limits for any long 2-lane tangent location. If this is verified by the passing study data, the analysis of which is nearing completion, it will be possible to determine whether or not a certain sight distance acts as a restriction to the free movement of traffic and the degree of restriction imposed at various hourly traffic volumes for highways with different free-speed distributions. This, in turn, will make possible the establishment of definite working capacities for any alinement based on the traffic density that is satisfactory for a tangent alinement.

Until the sight distances that must be available for safe drivers to pass vehicles going at a given speed in face of approaching vehicles traveling at various speeds are obtained from the analysis of the passing study data no definite values can be given for the practical traffic capacities of highways having numerous short sight distances at which none or only the slowest of the slow-moving vehicles can be passed. Each place on a highway where all passings are restricted for a certain distance places a restriction on the higher-speed passings for a greater distance.

It may be possible for a highway with alinement that restricts drivers from passing vehicles traveling 40 miles per hour 50 percent of its entire length to have a higher practical capacity than a highway with alinement that restricts all types of passings only 10 percent of its entire length. With 800 vehicles per hour as the practical capacity of a tangent highway, sight distances that prevent vehicles going 40 miles per hour from being passed would not reduce the capacity of a highway with free speeds as shown by figure 28, while the practical capacity of a highway with an alinement that prevented all passings over 10 percent of its length could not exceed 750 vehicles per hour to allow the same freedom from congestion as on a tangent section. The traffic density of 750 vehicles per hour was calculated by using the following formula:

$$V_R=\frac{2{,}000\ V_T\ (1-R)}{2{,}000-V_T R}$$

where

V_R = practical capacity in vehicles per hour on alinement where all passings are restricted for some distance,

V_T = practical capacity in vehicles per hour on tangent alinement, and

R = ratio of the distance that all passings are restricted to the entire length of the highway.

$$V_R=\frac{2,000\times800\times0.9}{2,000-80}\text{ or 750 vehicles per hour.}$$

This formula was derived from previously presented facts; namely,

1. That the maximum possible capacity of a 2-lane tangent highway is 2,000 vehicles per hour and occurs when no passings can be made (figs. 6, 8, and 15).
2. That a straight-line relationship exists between the hourly traffic volume and the ratio of the desired to the actual number of passings that are made (fig. 15).
3. That the ratio between the desired and actual number of passings that can be made is an index of the freedom from congested conditions.

FLUCTUATIONS IN TRAFFIC FLOW IMPORTANT CONSIDERATION

The formula is correct only when all types of passings that are made on a tangent section can also be made on all portions of the highway except where no passings can be made. Actually, the alinement would restrict the higher-speed passings over sections exceeding the length of the sections where all passings are restricted so the practical capacity of a highway that restricts all passings for 10 percent of its length would generally be somewhat less than 750 vehicles per hour. However, the formula may also be used when such a condition exists by substituting the summation of the product of the restricted portions of the highway by the percentage of passings restricted, as determined by figure 29, for the values substituted for R in the equation.

For example, when all passings are restricted on 10 percent of the highway, passing a vehicle going between 30 and 35 miles per hour is restricted on an additional 5 percent, and passing a vehicle going over 35 miles an hour is restricted on an additional 10 percent, R would be $0.10+(0.35\times0.05)+(0.25\times0.10)$ or 0.143 for the free speed distribution shown by figure 28 and the practical capacity becomes 728 vehicles per hour. (At a volume of 800 vehicles per hour on the tangent section 35 percent of the vehicles passed are going between 30 and 35 miles per hour and 25 percent are going over 35 miles per hour.) However, the results of the special passing studies are needed before the portion of any alinement that partially restricts passing maneuvers can be determined accurately.

The capacity figures thus far presented refer to hourly volumes for rural highways. Since there is a large variation in seasonal, daily, and hourly traffic volumes on any highway, it is not economical to design for the highest hourly density during the year. From an analysis [7] of data from several hundred automatic traffic recorders on rural highways throughout the country it has been found that relatively few drivers will be inconvenienced if a design is used that will provide for the traffic volume during the fiftieth highest hourly density during the year. Very little will be saved in the construction cost and a large number of drivers will be inconvenienced if a design value appreciably lower than the fiftieth highest hourly volume is used.

On an average, the fiftieth highest hourly density during the year on roads in northern States is about 15 percent of the annual average daily traffic. However, for certain roads this figure may be as high as 22 percent or as low as 12 percent. Two-lane roads with good alinements can therefore be expected to carry annual average 24-hour traffic volumes in excess of 3,500 vehicles, while congested conditions may occur on other 2-lane roads with poor alinement when the average annual volume exceeds 2,000 vehicles per day. Average traffic volumes are not a true index of the number of lanes necessary to handle traffic efficiently at a particular location. The type of traffic and the variation in traffic flow must be considered.

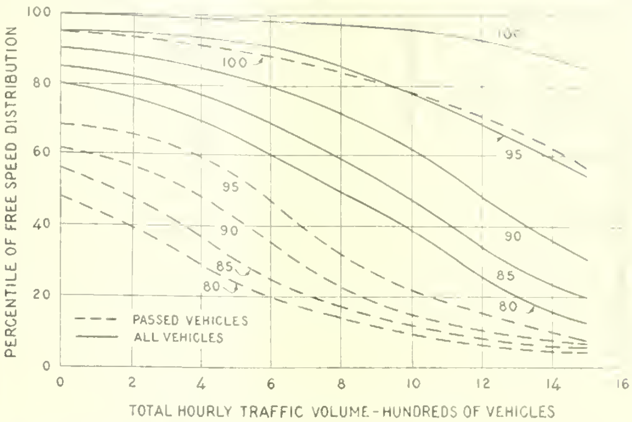

FIGURE 29.—RELATION BETWEEN HIGH PERCENTILE SPEEDS OF PASSED VEHICLES AND ALL VEHICLES AT VARIOUS TRAFFIC VOLUMES AND FREE SPEEDS.

CONCLUSIONS

The following statements give what are believed to be the most important conclusions to be drawn from the analysis of traffic data obtained at a large number of highway locations in several States.

1. While the theoretical and possible capacities of a highway are absolute values, the practical working capacity of a highway is a relative value that will vary for different local conditions.
2. The maximum traffic volumes that can be considered practical working capacities for 2-, 3-, and 4-lane rural highways are 800, 1,400, and 2,800 vehicles per hour, respectively.
3. The lower the hourly volume that is used as the practical capacity of a 2-lane road, the greater will be the difference between the practical capacities of 2-, 3-, and 4-lane roads with the same general alinement.
4. The design of a highway to handle traffic efficiently must be based on actual practices of all drivers in relation to the traffic on the highway rather than on the movement of individual vehicles over the highway.
5. Sight distances that restrict passing maneuvers on 2-lane highways vary with the traffic volume on the highway. Economical sight distances are a function of the traffic density.
6. The most important information needed to complete the study of practical working capacities for 2-lane highways with numerous nonpassing sight distances is the clear road space that drivers require before they will attempt to pass vehicles traveling at various speeds. Compared to this information, the time and distance requirements arrived at by assuming possible performance of vehicles and drivers are of little value.

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[7] Applications of Automatic Traffic Recorder Data in Highway Planning, by L. E Peabody and O. K. Normann. PUBLIC ROADS, January 1941.

"PUBLIC ROADS" TO APPEAR QUARTERLY IN FUTURE

Beginning with the next issue, PUBLIC ROADS will be issued as a quarterly rather than a monthly magazine. This course has been decided on as an economy in publication for the duration of the war and also because personnel formerly engaged in making highway research investigations have been assigned work directly connected with the war effort. The next issue (No. 5 of volume 23) will be for the July, August, and September, 1942, quarter and will be published in August.

Those desiring to obtain PUBLIC ROADS can do so by sending $1 for 12 issues (foreign subscriptions $1.50) or 10 cents per single copy, to the Superintendent of Documents, Government Printing Office, Washington, D. C.

Subscriptions already placed with the Superintendent of Documents will be extended to include the number of issues originally subscribed for.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF MAY 31, 1942

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$6,535,684	$3,293,985	228.2	$4,198,949	$2,284,183	115.2	$198,900	$99,450	0.2	$2,503,898
Arizona	1,874,458	1,412,624	73.8	1,093,598	884,779	41.6	238,615	188,508	1.7	1,814,393
Arkansas	3,666,578	1,675,354	57.9	1,146,936	572,392	59.5	206,581	124,149	1.7	1,700,402
California	9,269,588	5,242,912	153.6	5,011,061	3,448,485	58.5	1,518,357	1,072,374	39.8	3,153,054
Colorado	2,382,364	1,353,127	155.2	3,838,626	2,461,491	160.8	829,453	468,309	28.7	2,095,089
Connecticut	1,419,554	696,181	17.1	1,988,467	957,191	23.1	550,734	357,260	5.9	975,677
Delaware	595,356	345,071	17.1	428,811	210,460	9.0	268,040	134,020	8.4	1,491,402
Florida	1,342,189	698,897	66.7	3,492,278	2,027,851	52.9	150,000	112,500	.8	2,915,875
Georgia	3,085,217	1,526,545	111.7	7,875,091	4,102,996	290.8	2,765,380	1,382,690	117.6	6,704,986
Idaho	1,915,201	1,262,028	95.4	1,154,204	817,839	61.4	376,987	294,038	19.4	1,747,003
Illinois	4,124,984	2,003,325	89.5	7,388,745	4,292,602	125.3	2,762,528	1,680,096	22.1	5,390,859
Indiana	4,632,031	2,134,213	75.1	7,369,543	3,604,506	107.1				2,895,401
Iowa	4,220,074	2,043,789	184.5	4,398,037	1,884,495	123.0	133,767	32,100	5.6	2,310,531
Kansas	4,862,010	2,519,870	272.6	6,308,370	3,418,914	259.2	637,126	322,725	33.0	4,584,419
Kentucky	4,466,081	2,208,661	154.9	5,737,785	2,725,922	110.8	2,897,458	1,605,022	30.3	1,353,420
Louisiana	1,709,397	890,773	39.6	1,175,515	810,697	22.4	3,077,111	1,609,627	60.7	3,921,449
Maine	954,941	475,405	26.8	2,053,632	1,050,642	27.8	125,910	74,780	.5	1,095,000
Maryland	3,625,267	1,811,529	30.2	2,636,909	1,053,654	14.3	900,000	528,750	4.3	1,058,457
Massachusetts	2,770,017	1,387,639	19.2	1,915,968	1,021,145	13.7	1,170,360	581,593	8.4	3,824,051
Michigan	8,637,309	4,207,058	174.3	3,382,148	2,260,949	55.5	1,597,300	1,020,875	9.5	2,475,111
Minnesota	4,686,910	2,385,416	384.8	9,589,412	5,285,671	391.5	113,720	56,860	10.0	2,870,276
Mississippi	5,933,098	2,906,318	315.4	3,155,024	1,585,062	186.8	346,700	256,175	5.3	2,028,403
Missouri	5,589,688	2,877,438	173.1	10,256,772	6,005,148	184.6	2,145,532	1,131,492	25.2	3,365,594
Montana	2,471,129	1,482,070	138.1	3,887,470	2,399,603	181.5	210,734	119,823	26.6	4,276,700
Nebraska	3,560,106	1,748,959	346.2	4,603,229	2,330,504	408.3	629,763	454,687	36.1	4,065,412
Nevada	2,211,680	1,886,038	110.6	911,064	796,018	26.0	962,769	870,805	24.6	660,807
New Hampshire	337,058	213,772	5.9	1,700,559	1,148,668	20.2				691,062
New Jersey	3,613,528	1,767,619	29.6	2,374,192	1,254,616	13.2				2,984,381
New Mexico	1,870,408	1,236,746	125.0	920,899	701,088	51.1	29,346	24,078	5.0	2,651,571
New York	9,503,520	4,722,722	126.7	8,683,631	5,558,186	86.7	429,000	193,500	6.4	5,215,966
North Carolina	4,737,420	2,556,296	188.3	2,167,664	1,144,767	98.0	561,756	283,902	8.8	3,674,957
North Dakota	3,224,478	1,874,388	287.6	2,810,655	1,635,782	223.2	2,173,990	1,090,365	186.2	4,020,406
Ohio	9,976,118	5,459,242	97.9	14,665,022	7,632,574	84.1	1,924,480	847,988	19.2	2,935,007
Oklahoma	2,651,649	1,383,859	122.3	2,692,322	1,500,534	64.9	1,613,930	928,149	39.7	5,960,800
Oregon	2,826,493	1,801,454	73.0	3,285,375	1,716,916	76.5	1,271,346	988,690	36.7	718,935
Pennsylvania	10,852,021	5,354,296	107.0	10,083,473	5,321,771	74.4	1,229,936	780,404	9.6	4,208,632
Rhode Island	1,194,495	678,166	10.0	1,046,898	669,630	5.0	479,631	239,757	2.6	601,761
South Carolina	2,987,866	1,606,085	109.5	3,402,258	2,130,846	83.5	1,138,590	747,575	20.0	971,996
South Dakota	2,373,781	1,399,681	287.9	4,709,133	3,116,405	533.3	545,110	325,580	59.3	2,882,579
Tennessee	4,616,521	2,592,911	110.5	4,744,348	2,743,791	102.6				3,454,968
Texas	14,171,091	7,026,419	676.5	10,265,021	4,894,258	305.9	1,215,090	609,635	35.6	8,787,008
Utah	1,251,036	939,893	53.8	1,838,587	1,393,120	46.7	33,518	26,800	6.8	1,229,710
Vermont	802,020	449,293	28.8	1,193,326	715,241	20.7	36,906	18,453	.3	395,551
Virginia	4,280,307	1,981,873	81.1	3,271,128	1,780,479	51.0	35,490	17,745	.4	2,745,066
Washington	1,808,560	987,042	26.6	2,514,659	1,362,842	30.5	204,707	109,600	3.4	2,077,996
West Virginia	3,318,393	1,674,884	57.2	2,259,388	1,206,153	25.3	1,261,269	773,589	6.5	1,322,960
Wisconsin	2,178,714	1,133,634	92.7	6,445,284	4,177,243	175.6				3,656,668
Wyoming	1,439,453	1,021,694	148.4	1,773,023	1,317,527	123.5				1,278,336
District of Columbia	1,055,468	541,238	3.5	214,289	127,960	.9	1,147,456	665,688	1.5	44,668
Hawaii	141,835	75,057	2.8	1,115,128	759,983	10.9	499,587	480,115	5.2	1,477,578
Puerto Rico	409,945	238,835	3.9	2,504,688	1,237,102	21.3	37,725	27,994	.2	652,951
TOTALS	188,253,089	99,192,324	6,368.1	201,678,594	113,540,681	5,439.6	40,682,688	23,758,315	979.8	135,919,182

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF MAY 31, 1942

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$1,392,054	$691,362	62.5	$685,882	$362,410	31.1	$40,801	$19,550	0.1	$586,086
Arizona	255,110	184,247	14.1	138,065	101,042	6.8				513,118
Arkansas	629,977	237,235	33.1	483,854	241,861	26.5	171,789	84,297	6.0	251,526
California	1,062,396	656,498	18.3	723,924	554,248	7.6	66,000	47,400	1.8	982,940
Colorado	209,672	112,570	20.9	131,800	73,914	3.2	152,386	35,323	5.0	502,702
Connecticut	298,035	136,331	6.1	266,247	115,937	4.8				199,604
Delaware	164,535	79,371	.4	141,464	70,732	11.9	102,873	37,618	3.9	246,037
Florida	1,042,471	519,285	11.6	292,018	153,304	6.2				398,953
Georgia	557,275	263,474	43.4	1,193,931	689,315	74.9	400,257	200,129	43.7	1,126,812
Idaho	299,357	172,516	26.1	225,220	157,021	7.7	38,149	23,587	2.8	285,992
Illinois	1,087,608	515,800	59.4	1,264,884	632,442	65.1				896,158
Indiana	586,368	285,054	39.8	1,283,255	608,471	51.0	88,800	44,400	3.1	920,323
Iowa	658,901	309,137	165.1	558,583	251,770	79.5	126,376	57,350	29.1	584,790
Kansas	865,347	436,593	112.1	1,790,788	900,372	92.1	238,353	119,177	29.7	1,031,750
Kentucky	1,154,545	321,329	83.2	1,249,010	417,668	37.2	74,213	24,400	5.6	324,331
Louisiana	539,462	265,678	20.6	7,700	3,850		289,362	138,761	21.5	666,650
Maine	89,922	44,961	4.6	223,818	111,909	9.6	16,850	2,714	.5	160,925
Maryland	579,966	289,808	20.7	226,410	113,205	1.8				345,564
Massachusetts	137,789	101,569	4.1	700,733	390,183	10.1				526,805
Michigan	1,299,446	633,000	81.4	760,498	380,249	29.5	245,970	122,985	1.3	654,241
Minnesota	1,613,984	792,692	217.1	1,021,469	516,473	91.8	279,496	139,348	26.8	592,011
Mississippi	851,015	424,488	47.7	1,494,546	709,614	59.0				376,864
Missouri	481,174	237,794	55.8	886,338	425,581	91.1	186,244	63,193	32.4	1,045,734
Montana	374,571	210,387	58.6	292,724	170,722	31.0	13,569	7,715	4.5	904,396
Nebraska	457,436	225,708	57.3	404,827	207,640	58.7				694,594
Nevada	331,931	266,129	28.6	92,429	60,188	4.6				234,872
New Hampshire	156,054	95,143	4.7	239,437	163,488	3.6				88,975
New Jersey	518,759	254,877	9.6	437,305	245,055	13.8				571,260
New Mexico	566,935	357,245	47.7	262,325	169,211	19.5				250,178
New York	956,940	468,240	28.8	1,160,340	620,808	14.9	245,600	122,800	.9	1,088,867
North Carolina	331,976	170,672	34.8	522,407	278,748	36.3	150,810	60,330	6.8	654,350
North Dakota	29,802	15,664	2.4	7,382	7,382		808,050	793,860	42.7	755,172
Ohio	1,780,920	886,359	57.6	901,610	498,775	14.5	157,640	78,820	2.9	1,336,590
Oklahoma	400,166	210,874	27.6	27,872	14,715	4.0	1,240,706	655,126	75.9	905,068
Oregon	547,387	298,453	50.0	395,568	230,087	20.3	30,482	18,000	1.3	322,972
Pennsylvania	2,004,228	986,137	37.6	513,989	256,587	10.4	73,588	36,794	1.8	786,814
Rhode Island	231,189	119,932	2.6	11,884	9,692		104,895	52,447	1.0	78,463
South Carolina	773,386	307,260	54.6	221,700	79,945	.5				398,326
South Dakota	37,125	22,152	15.2	9,056	9,056		1,143,430	1,047,600	114.5	757,118
Tennessee	689,893	343,243	16.8	1,141,522	580,761	43.2	158,042	79,021	4.5	747,642
Texas	1,577,788	769,471	165.0	512,790	250,470	34.9				2,237,214
Utah	204,014	136,160	20.3	136,491	88,790	3.5	38,932	28,724	.8	308,319
Vermont	40,708	18,109	1.2	222,515	117,328	8.9				53,580
Virginia	403,187	189,037	16.2	348,516	156,180	4.6	23,400	11,700	3.8	679,170
Washington	391,635	228,446	23.7	349,818	147,362	7.8				410,431
West Virginia	428,549	215,282	19.9	285,664	142,219	6.3				510,286
Wisconsin	947,262	475,932	42.7	1,460,828	668,455	46.4	35,787	17,000	.8	580,134
Wyoming	365,617	160,649	18.8	508,423	218,112	34.3				218,048
District of Columbia	79,178	39,035	.9							163,600
Hawaii	1,158	1,096		1,279	1,279					334,875
Puerto Rico	170,027	77,541	7.5	73,642	38,810	3.1				217,968
TOTALS	30,704,230	15,260,025	1,998.8	26,292,760	13,413,436	1,223.6	6,742,850	4,170,169	475.5	29,509,198

U. S. GOVERNMENT PRINTING OFFICE: 1942

PUBLICATIONS of the PUBLIC ROADS ADMINISTRATION

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Agency and as the Agency does not sell publications, please send no remittance to the Federal Works Agency.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1932. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937 10 cents.

Report of the Chief of the Bureau of Public Roads, 1938. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1939. 10 cents.

Work of the Public Roads Administration, 1940, 10 cents.

Work of the Public Roads Administration, 1941, 15 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . . The Results of Physical Tests of Road-Building Rock. 25 cents.

No. 191MP. . Roadside Improvement. 10 cents.

No. 272MP. . Construction of Private Driveways. 10 cents.

No. 279MP. . Bibliography on Highway Lighting. 5 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. 60 cents.

Highways of History. 25 cents.

Specifications for Construction of Roads and Bridges in National Forests and National Parks. 1 dollar.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.

No. 265T. . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Public Roads Administration upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP. . Bibliography on Highway Safety.

House Document No. 272 . . . Toll Roads and Free Roads.

Indexes to PUBLIC ROADS, volumes 6–8 and 10–21, inclusive.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

REPORTS IN COOPERATION WITH UNIVERSITY OF ILLINOIS

No. 303. . . Solutions for Certain Rectangular Slabs Continuous Over Flexible Support.

No. 304. . . A Distribution Procedure for the Analysis of Slabs Continuous Over Flexible Beams.

No. 313. . . Tests of Plaster-Model Slabs Subjected to Concentrated Loads.

No. 314. . . Tests of Reinforced Concrete Slabs Subjected to Concentrated Loads.

No. 315. . . Moments in Simple Span Bridge Slabs With Stiffened Edges.

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF MAY 31, 1942

STATE	COMPLETED DURING CURRENT FISCAL YEAR		NUMBER			UNDER CONSTRUCTION		NUMBER			APPROVED FOR CONSTRUCTION		NUMBER			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$154,270	$153,734	2	4	2	$416,125	$412,903	6	2	2	$230,835	$230,835	4		5	$918,999
Arizona	184,378	184,361	1	1		138,529	129,839		1	2	4,095	4,095			1	230,684
Arkansas	474,307	471,932	5		9	169,829	168,116	1		2	18,911	18,515			6	676,286
California	1,036,632	844,943	7	1	7	676,477	676,477	3			3,775	3,775			1	2,319,499
Colorado	52,743	52,704	1		1	623,402	623,402	6		2	14,914	14,914			7	745,764
Connecticut	166,222	165,415	2			61,712	60,676		1		231,374	222,740		1		540,683
Delaware	89,725	89,725	1		1	191,599	189,867	1	1		508,406	321,785	2			179,564
Florida	120,961	120,961			20	843,067	840,887	8		6	82,492	82,432			14	871,382
Georgia	681,733	681,525	7	2	7	994,222	994,222	6	6	13	785,985	785,985	2	4	9	1,505,777
Idaho	113,531	109,262	1		3	236,450	227,778	3	1		4,190	4,190			2	427,835
Illinois	851,316	601,160	2		97	1,649,097	1,565,901	8	1	2	387,209	387,009	1	1	30	2,541,298
Indiana	624,117	618,249	6		31	461,883	461,883	2	1	14	95,230	95,230			28	1,148,196
Iowa	378,416	372,400	3	1	36	1,519,599	1,262,426	11	2	23	114,376	112,120			29	626,079
Kansas	93,276	83,870	2		8	742,064	742,064	9		3	141,011	141,011	3		6	1,304,512
Kentucky	1,148,734	1,146,371	9	1	1	402,849	402,849	4								411,382
Louisiana	14,170	14,170			2	586,220	586,220	8			474,630	473,463	4		2	924,798
Maine	10,383	10,383			3	364,948	364,948	2		1						286,599
Maryland	500,850	469,057	2	2	12	874,458	730,660	4		4	30,175	30,175			6	405,924
Massachusetts	346,270	335,829	1		1	791,642	790,770	5	2		763,830	763,830	2	1		1,250,554
Michigan	1,416,063	1,397,726	3	4	25	562,166	562,166	2		2	866,629	836,468	2		7	395,559
Minnesota	641,332	640,745	5	4	2	977,175	977,175	5	4	6	4,985	4,985			1	1,238,518
Mississippi	253,874	253,874	2	1	10	862,718	862,718	10			6,200	6,200			2	646,344
Missouri	593,138	414,702	2	2		1,719,957	1,442,973	6	2		203,565	203,565		2		1,480,365
Montana	141,549	141,549	2		2	99,778	99,778	1								733,051
Nebraska	484,061	482,941	7		22	837,830	837,830	16			13,020	13,020			3	473,749
Nevada	126,306	121,955	2		3	57,946	57,946	2			22,635	22,635			7	196,853
New Hampshire	287,335	287,036	4	2	5	74,598	74,273	2								313,961
New Jersey	852,812	834,882	4	1	2	622,904	497,354	3	1		354,985	295,560	1	2		956,807
New Mexico						71,000	71,000	1			259,103	252,068	3		1	519,348
New York	2,422,733	2,364,650	2	13		1,982,146	1,941,007	3	8		502,645	464,285		1		3,729,719
North Carolina	626,618	623,740	2	6	34	187,142	187,142	1	1	4	33,695	33,695			8	1,407,650
North Dakota	289,022	287,819	5			470,895	470,895	5								872,117
Ohio	1,557,247	1,542,030	8	1	12	2,752,053	2,428,331	10		1	684,590	480,460	2		2	1,423,700
Oklahoma	256,234	250,950	2		35	880,359	878,439	6		3	392,744	354,707	3		3	1,432,840
Oregon	423,249	358,685	4		3	281,499	228,712	2			13,433	13,433			4	399,969
Pennsylvania	1,907,614	1,876,905	14	1		3,140,659	3,126,130	15			235,355	235,355	1			2,526,669
Rhode Island	205,241	205,241		1		3,655	3,655									273,921
South Carolina	529,339	510,274	7	2	25	84,269	69,128	2	1		211,228	96,095		2	6	992,889
South Dakota	511,729	511,618	13		9	515,821	499,871	9			41,200	41,200	2			829,404
Tennessee	371,045	359,151	4	3		1,256,182	1,256,182	6	1		427,478	427,478	1	1	4	680,015
Texas	1,674,280	1,654,917	21			1,025,741	1,006,216	8		4	4,350	4,350			1	2,376,450
Utah	86,598	85,856	2		28	43,874	43,874	1		1	57,831	57,831			22	359,638
Vermont	17,426	16,698			4	322,869	293,090	2								114,657
Virginia	566,180	566,180	4	4	4	347,127	327,167	3	1	1						965,269
Washington	395,005	394,758	3	2	4	1,041,232	344,558	5			2,478	2,478			1	441,484
West Virginia	253,143	247,512	3		6	658,477	658,477	6	1	2	447,830	447,830	1		1	328,286
Wisconsin	477,790	460,449	2		41	560,581	559,593	3	2	5	12,687	12,687			2	1,675,841
Wyoming	481,187	466,531	6		1	1,974	1,974				8,417	8,416			6	430,012
District of Columbia	3,655	3,655				298,213	273,744		1							103,351
Hawaii	189,832	189,811	2			211,977	211,462	2								282,756
Puerto Rico	103,629	102,980	1			780,618	772,706	11								363,914
TOTALS	25,187,300	24,181,871	188	59	518	34,477,607	32,297,454	235	41	103	8,698,521	8,006,905	34	15	227	46,280,921

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

FEDERAL WORKS AGENCY
PUBLIC ROADS ADMINISTRATION

VOL. 23, NO. 5 JULY-AUGUST-SEPTEMBER 1942

ENGINEERS MEASURE THE STRENGTH OF A MONOLITHIC BRICK TEST SLAB

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PUBLIC ROADS

A Journal of Highway Research

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D. M. BEACH, *Editor*

Volume 23, No. 5

July–August–September 1942

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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TESTS OF MONOLITHIC BRICK PAVEMENT SLABS

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by **W. F. KELLERMANN**, Materials Engineer

Some years ago monolithic pavements were constructed by placing the brick in the fresh concrete of the base and filling the joints with mortar. This type of pavement was discarded in favor of the conventional two-course brick pavement (1) because of trouble experienced with blow-ups which resulted from the absence of adequate expansion joints and (2) because the riding qualities were not as good as those of other high type pavements.

This report describes a recent development in the technique of constructing monolithic brick pavements and discusses the results of a series of tests made to determine the quality of pavement slabs so constructed as compared with the quality of pavement slabs constructed entirely of concrete.

The results of these tests have indicated that monolithic brick slabs, when tested with the concrete in tension, developed higher flexural strengths than comparable slabs of plain concrete. However, when the brick surface was placed in tension, the monolithic brick slabs developed much lower strengths than comparable slabs of plain concrete. In the latter case the angle at which the brick courses were placed with respect to the loading knife edge of the testing machine had an effect upon the results obtained.

In tests made with the brick surface in tension, the bond between the concrete and the brick was not affected by 100 applications of load equal to 50 percent of the ultimate load.

In the construction of brick pavements with concrete bases it has been customary during recent years to lay the brick in a bedding course of sand, or a mixture of sand and bitumen, that has been spread on the hardened concrete base, and to fill the joints between the brick with bitumen. With this type of construction, known as the two-course type, the brick wearing course could not be expected to add appreciably to the strength of the concrete base.

A number of years ago monolithic brick pavements were constructed by laying the brick in the fresh concrete of the base, rolling with a light roller, and filling the joints with cement grout. At that time the need for expansion joints was not fully appreciated and much trouble was experienced with blow-ups caused by expansion. Also the riding qualities of monolithic brick pavements did not compare favorably with the riding qualities of other high-type pavements. As a result, the monolithic type fell into disuse and the two-course type was used exclusively in brick pavement construction.

With the introduction of surface vibration as a means of compacting the concrete in pavement slabs, new possibilities were presented for the construction of monolithic brick pavements. Consequently, several monolithic brick pavements have been built in which the brick have been embedded in the concrete base by means of high-frequency surface vibrators, rather than by rollers.

The procedure is first to build the usual concrete base using a finishing machine equipped with surface vibrators. The brick are then immediately placed on the fresh concrete and vibrated with a machine designed especially for the purpose. This machine is built along the same general lines as the conventional finishing machine except that in place of the conventional screeds there is a special screed consisting of a series of steel channels. These channels are vibrated as they ride over the brick, with the result that the brick are brought to proper elevation, at the same time being forced down about ⅛ inch into the concrete. A thin sand-cement grout is then squeegeed over the brick surface, filling the joints. After the grout has taken its initial set, a second application of somewhat stiffer grout is applied. A final mortar finish over the entire surface is obtained by use of a burlap drag. It is claimed that this type of construction eliminates the objectionable features of the old-type monolithic pavement and that pavements constructed by this method are superior in many respects to the two-course type.

TWO SERIES OF TESTS MADE

In order to determine the flexural strengths of monolithic brick slabs constructed in this manner as compared with those of plain concrete compacted by surface vibration, two series of laboratory tests (series A and series B) were initiated in which the following variables were studied:

SERIES A

1. Effect of total depth of slab.
2. Effect of testing the top surface of the slab, as cast, in tension as compared with the bottom surface.
3. Effect of angle of brick courses with respect to the longitudinal axis of slab.
4. Effect of repeated loadings.

SERIES B

1. Effect of testing the top surface of the slab in tension as compared with the bottom surface.
2. Effect of angle of brick courses with respect to longitudinal axis of slab.
3. Effect of richness of concrete mix.

The brick used in these tests were the conventional 3- by 4- by 8½-inch vertical fiber lug brick normally used in pavement construction. Grout used as a filler was proportioned 1:2 by dry loose volume with sufficient water to give the desired consistency. Tables 1 and 2 give the physical properties of the aggregate while the mix data for the concrete and the grout are given in tables 3 and 4, respectively. Different cements were used in the two series. Results of strength tests on the two cements are as follows:

	Tensile strength, lb per sq. in.
Series A:	
At 7 days	405
At 28 days	485
Series B:	
At 7 days	380
At 28 days	435

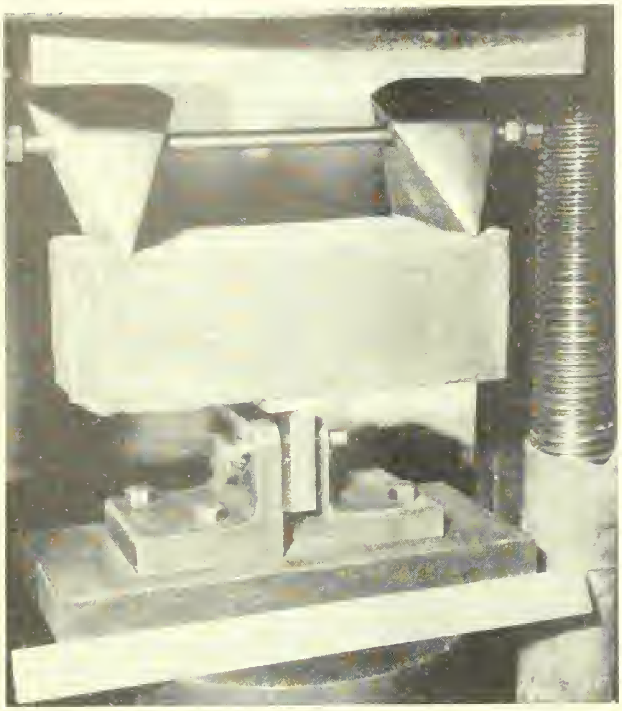


Figure 1.—Set-up Used in Determining Flexural Strength of Paving Brick.

Table 1.—*Physical properties of aggregates for monolithic brick pavement slabs*

Aggregates	Character of aggregate	Bulk specific gravity	Weight per cubic foot		Voids, dry rodded	Absorption	Los Angeles abrasion loss [1]
			Solid	Dry rodded			
For concrete:			*Pounds*	*Pounds*	*Percent*	*Percent*	*Percent*
Sand, series A	Quartz	2.59	162	106	35	1.00	....
Sand, series B	do	2.57	160	105	34	1.50	....
Gravel	do	2.58	161	107	34	.99	28.0
For grout: Sand	do	2.60	162	[2] 90	[2] 44	1.00	....

[1] Grading A used. [2] Dry loose.

Table 2.—*Grading of aggregates for monolithic brick pavement slabs*

Sieve size	Percentage retained			
	Mortar sand	Concrete sand		Gravel
		Series A	Series B	
1½-inch	0	0	0	0
1-inch	0	0	0	38
¾-inch	0	0	0	57
⅜-inch	0	0	0	86
No. 4	0	1	1	100
No. 8	0	13	16	100
No. 16	0	25	31	100
No. 30	17	42	54	100
No. 50	65	74	83	100
No. 100	94	95	96	100
Fineness modulus	1.76	2.50	2.81	7.43

Results of tests made on the brick are given in table 5. The set-up for determining the flexural strength of the brick is shown in figure 1.

In this report the term "pavement concrete" is used to designate concrete containing 6 sacks of cement per cubic yard, which is a class commonly used in pavements. The term "base concrete" is used to designate concrete containing 4¾ sacks of cement per cubic yard, which is a class frequently used in the construction of bases.

Pavement concrete was used in series A, with sufficient water to produce a slump of approximately 1 inch.

Test specimens consisted of slabs approximately 26 inches wide by 60 inches long and 7, 8, and 10 inches in depth. Both plain concrete and monolithic brick slabs were cast for each thickness.

Table 3.—*Data on concrete mixes for monolithic brick pavement slabs*

Proportions by weight, pounds	W/C by volume	W_c by volume ([1])	b/b_0 ([2])	Mortar voids ratio	Sand in total aggregate, by weight	Cement factor	Slump
					Percent	*Sacks per cu. yd.*	*Inches*
Series A: 94 . 173 : 367	0.65	0.145	0.77	1.87	32	6.0	1.0
Series B:							
94 : 184 : 356	.65	.145	.74	2.02	34	6.0	.75
94 : 253 : 451	.84	.148	.74	2.04	36	4.75	.75

[1] Water per unit volume of concrete.
[2] Apparent volume of coarse aggregate per unit volume of concrete, dry rodded basis.

Table 4.—*Data on grout used in monolithic brick pavement slabs, series A and B*

Proportions		W/C by volume	
By dry loose volume	By weight	First grout	Second grout
1:2	*Pounds* 94:180	1.01	0.86

Table 5.—*Physical properties of paving brick used in monolithic brick pavement slabs*

Test	Average [1]	Maximum	Minimum
Absorption.........percent	4.54	5.67	3.54
Specific gravity	2.36	2.39	2.32
Standard rattler.........percent	[2] 13.6	....	....
Modulus of rupture—pounds per square inch	2,250	2,540	1,980
Crushing strength.........do	18,380	21,690	12,830

[1] Each result is the average for 10 tests except where otherwise noted.
[2] One test.

The plain concrete slabs were constructed in the following manner: The form was filled and an excess of concrete placed on top in order to allow for subsidence. Two complete passes were then made over the entire surface with a platform vibrator operating at a frequency of 7,000 pulsations per minute. The slab was then struck off with a wooden screed. In the case of the monolithic brick slabs the form was filled to within 2½ inches of the top and the concrete vibrated in the same manner. The brick were then set in the fresh concrete and vibrated in the same manner as for the plain concrete slabs (see fig. 2). This procedure resulted in twice as much vibration of the concrete in the monolithic as in the plain concrete slabs. It was adopted because it followed actual practice for the two different types of construction in the field.

After vibrating, a 1:2 mortar grout sufficiently thin to penetrate the joints was squeegeed over the surface. As this grout stiffened it also subsided, leaving the joints low. A second and somewhat thicker grout was then applied to fill the joints. The final finish was with a burlap drag, leaving a thin mortar top over the entire surface.

All slabs were kept under wet burlap for 2 days, after which they were removed from the molds and cured in a fog room at 70° F. until tested at 29 days. In the monolithic brick slabs, three different arrangements of the brick were used. In one case the brick were laid with the courses parallel to the longitudinal axis of the slab. In the second case they were laid transversely to the longitudinal axis of the slab; while in the third case they were laid at an angle of 45°.

One group of slabs was tested with the bottom as cast in tension; the other group was tested with the top or brick surface in tension. Those slabs tested with the bottom in tension were made in groups of three, of which one was a plain concrete slab and two were monolithic brick. When testing the concrete slab and one of the monolithic slabs, each slab was loaded to failure with one application of loading. This method of loading will be referred to as instantaneous loading. The other monolithic slab was given 100 applications of a load equivalent to one-half the ultimate load obtained in the test of the first monolithic slab. The specimen was then loaded to failure. This procedure was followed in order to determine whether 100 repetitions of 50 percent of the ultimate load would produce any separation of the brick surface from the concrete base. Table 6 gives a complete outline of the procedure together with the strength results. All slabs were tested in a portable testing machine, using a pair of calibrated, heat-treated steel beams as a load-measuring device. The cover illustration shows a set-up of the testing equipment. The specimens were tested as simple beams with the load applied at the center of a 54-inch span.

TABLE 6.—*Summary of results of flexure tests on monolithic brick pavement slabs, series A* [1]

Structure	Nominal depth of slab	Surface in tension [2]	Direction of knife edge of applied load [3]	Coarse aggregate broken	Flexural strength,[4] age 29 days	Strength ratio [5]
	Inches			*Percent*	*Lb./sq. in.*	*Percent*
Concrete	7	Bottom		55	730	100
Monolithic brick	7	do	Parallel	60	868	119
Do	7	do	do.[6]	70	841	115
Concrete	8	do		60	741	100
Monolithic brick	8	do	Parallel	65	840	113
Do	8	do	do.[6]	55	788	106
Concrete	10	do		50	728	100
Monolithic brick	10	do	Parallel	65	859	118
Do	10	do	do.[6]	60	884	121
Concrete	7	Top		55	797	100
Monolithic brick	7	do	Perpendicular	65	636	80
Do	7	do	45° angle	70	535	67
Do	7	do	Parallel	45	341	43
Concrete	8	do		60	845	100
Monolithic brick	8	do	Perpendicular	50	724	86
Do	8	do	45° angle	50	508	60
Do	8	do	Parallel	40	367	43
Concrete	10	do		50	776	100
Monolithic brick	10	do	Perpendicular	60	790	102
Do	10	do	45° angle	40	588	76
Do	10	do	Parallel	55	443	57

[1] Specimens 26 inches wide by 60 inches long (approximately); tested with center loading on 54-inch span after 29 days of moist curing.

[2] In all cases where the slabs were tested with the bottom surface in tension, each result is the average of two tests. All other values are individual test results.

[3] With respect to the line of continuous brick courses.

[4] Flexural strength computed with the neutral axis assumed at the center of the slab.

[5] Plain concrete slabs in each case were taken as standard.

[6] Given 100 repetitions of a load equal to 50 percent of the ultimate load of the corresponding slab and then tested to failure.

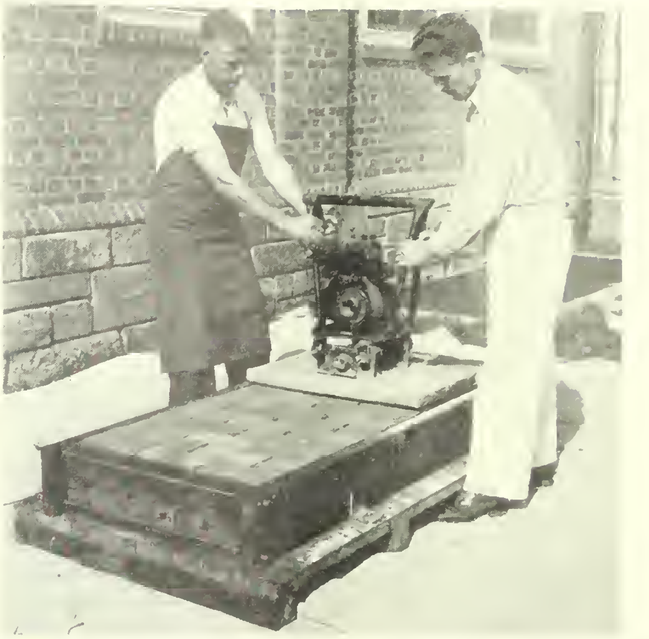


FIGURE 2.—VIBRATOR USED FOR COMPACTING PLAIN CONCRETE AND MONOLITHIC BRICK SLABS.

RICHNESS OF CONCRETE MIXTURE A VARIABLE IN SERIES B

In series B the same procedure of casting, curing, and testing was employed as in series A, except that in the brick slabs that were tested with the top, as cast, in tension, the brick were placed at an angle of 45° with respect to the longitudinal axis instead of in three different ways, as was done in series A. Also, all specimens were tested by instantaneous loading. Tests were made at 28 days instead of at 29 days as in series A.

One variable, not included in series A but studied in series B, was the effect of the richness of the concrete mix. In series A pavement concrete was used in all slabs, while in series B both pavement concrete and base concrete were used. This was done in order to compare the strength of monolithic brick slabs of base concrete directly with the strength of vibrated pavement concrete of the same total thickness. The effect of depth of slab was not studied in series B, all specimens being 8 inches thick.

Strength results are tabulated in table 6 and shown in graphical form in figure 3. In computing the values for modulus of rupture it was assumed that the neutral axis was at the center of the slab. It is realized that this may not be a correct assumption, particularly in the case of the monolithic brick slabs. However, for purposes of determining comparative load-carrying capacity, this procedure is justified, since comparisons are made between monolithic brick and plain concrete specimens of equal thickness. The same comparisons could be made with the total breaking loads.

Considering first the results plotted in figure 3 for those specimens tested with the bottom, as cast, in tension (brick surface in compression) the monolithic brick slabs gave higher values for computed moduli of rupture than did those of plain concrete. This was true for slabs of all thicknesses tested. The modulus of rupture of the monolithic brick specimens tested instantaneously averaged 17 percent higher than that of the plain concrete, while for the monolithic slabs given 100 repetitions of 50 percent loading the modulus of rupture averaged 14 percent higher than that of the plain concrete. It will be noted that in two out of three cases the monolithic brick slabs given 100 repetitions of 50 percent of the ultimate load and then loaded to failure gave slightly lower strengths than did

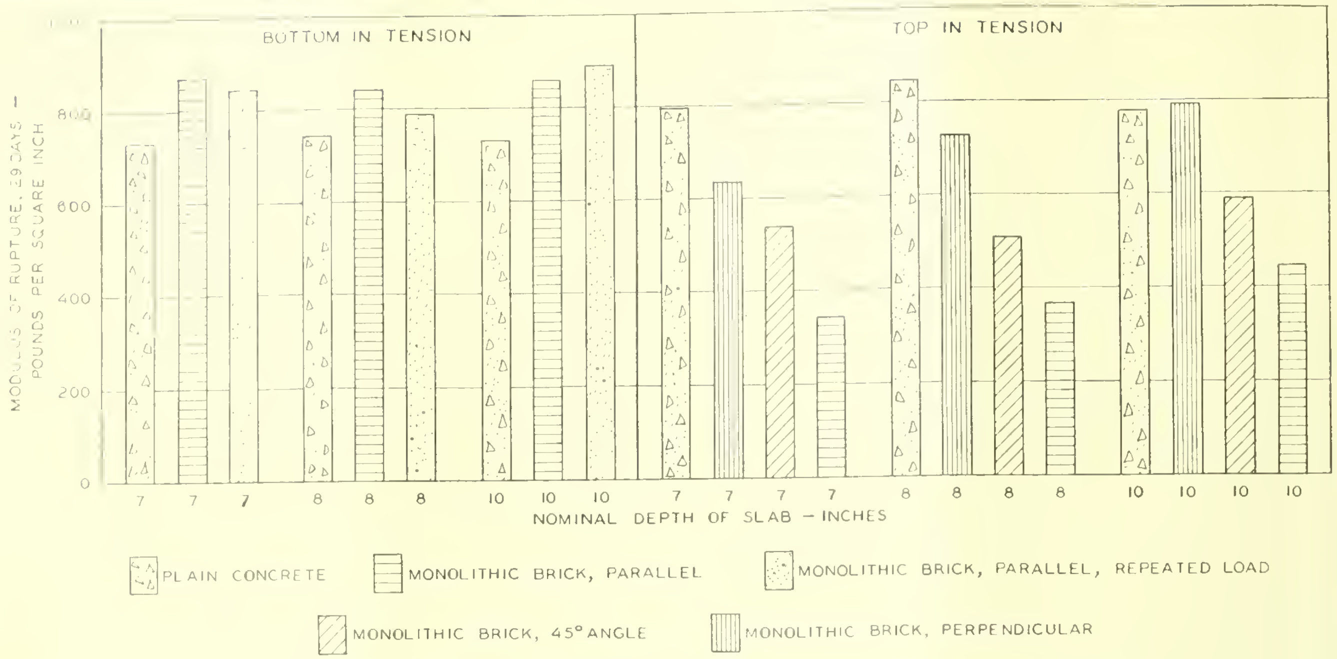


Figure 3.—Results of Flexure Tests on Monolithic Brick Pavement Slabs, Series A, at Age of 29 Days. "Parallel," "45° Angle," and "Perpendicular" Refer to the Direction of Knife Edge of Applied Load with Respect to the Line of Continuous Brick Courses. "Repeated Load" Indicates Slabs Were Given 100 Repetitions of a Load Equal to 50 Percent of the Ultimate Load of the Corresponding Slab and Then Tested to Failure.

the corresponding slabs tested with but one application of loading. It was anticipated that the repeated flexing of the slab might tend to destroy the bond between the brick surface and the concrete base but apparently this did not take place since the differences between the results obtained with the single loading and after repeated loading are within the limits of experimental error.

In all cases where the slabs were tested after repeated loadings, the percentage of brick broken at the section was less than in cases where they were tested with but one application of load. In the case of the 7-inch monolithic brick slabs tested by instantaneous loading the percentage of brick broken was 40 while for the companion slabs tested after repeated loading the percentage was 20. For the 8-inch slabs the differential was even greater whereas for the 10-inch slabs it was somewhat less.

Data for the slabs tested with the top surface, as cast, in tension are also plotted in figure 3. In this group no specimens were given repeated loadings. However, the brick were placed in three different directions so that there were four slabs in each group, one of plain concrete and three of monolithic brick. In each group the left bar represents the strength of the plain concrete while the second bar represents the strength of monolithic brick slabs in which the line of continuous courses of brick was perpendicular to the axis of the loading knife edge of the testing machine. In this case the maximum bending moment occurred at the center of the individual brick in three of the six brick courses in the cross section, while, due to the staggering of courses, the maximum bending moment occurred at the joint between brick and mortar in the other three cases. This is illustrated in figure 4–A, which shows a view of a fractured monolithic slab.

The third bar in the group gives the strength of monolithic brick slabs in which the direction of brick courses was at an angle of 45° to the axis of the loading knife edge. In this test failure occurred at the joint between the mortar and the brick (see fig. 4–B). The fourth bar in each group gives the strength of monolithic brick slabs in which the courses of brick were parallel to the axis of the loading knife edge. As was to be expected from this procedure, failure occurred at the joint between the brick and the mortar as shown in figure 4–C.

STRENGTH AFFECTED BY DIRECTION OF BRICK COURSES

The direction of the courses of brick had a marked influence upon the strength. Of the three ways of placing the brick, the highest strengths were obtained by placing the line of brick courses perpendicular to the axis of the loading knife edge, while the lowest strengths were obtained by placing the line of brick courses parallel to the axis of the loading knife edge. Brick courses placed at an angle of 45° to the loading knife edge gave intermediate results. Because of the failure in bond between the brick and grout, the monolithic brick slabs tested with the brick in tension gave much lower results than did the plain concrete slabs, the only exception being the 10-inch monolithic slabs with the brick courses perpendicular to the axis of the loading knife edge, which gave results slightly higher than those obtained with the plain concrete.

In summary, it was found that monolithic brick slabs tested with the concrete surface in tension gave results about 17 percent higher than those for plain concrete of equal thickness. On the other hand, when tested with the brick surface in tension and with the courses of brick set parallel to the axis of the loading knife edge, monolithic brick averaged only about 50 percent of the strength of plain concrete. The direction of the brick courses in the slabs had a marked effect in this test. Those slabs in which the courses of brick were set

(Continued on page 97)

VERTICAL CURVES FOR HIGHWAYS

BY THE DIVISION OF DESIGN, PUBLIC ROADS ADMINISTRATION

Reported by **D. W. LOUTZENHEISER**, Associate Highway Engineer

Length of vertical curve.—Any vertical curve on a highway should be as long as possible but the minimum length should be based on the sight distance requirements for the highway. The sight distances required are based on the assumed design speed for the highway. At any point on a highway the sight distance should be at least long enough that a vehicle traveling at the design speed can, upon sight of an object on the road, be brought to a stop before reaching it. Such sight distances are known as "nonpassing sight distances" and are not long enough on 2-lane roads for safe passing of vehicles in the same direction in the face of opposing traffic. Sight distances safe for passing on 2- and 3-lane roads are known as "passing sight distances." They rarely can be provided at all points, but they should be provided at frequent intervals.

The pamphlet "A Policy on Sight Distances for Highways," published by the American Association of State Highway Officials, presents a discussion of pertinent factors, calculations of and recommendations for minimum values of both nonpassing and passing sight distances for various highway design speeds. Tables 1 and 2 give the minimum lengths of vertical curve necessary for the various design speed-sight distance requirements. Reference is made to the sight distance policy for the numerical values of the various sight distances and formulas for calculation.

TABLE 1.—*Minimum length of vertical curves to provide minimum nonpassing sight distance*

A=algebraic difference of grades—percent	Length of vertical curve for assumed design speed of—				
	30 mi. per hr.	40 mi. per hr.	50 mi. per hr.	60 mi. per hr.	70 mi. per hr.
	Stations	*Stations*	*Stations*	*Stations*	*Stations*
2	0	0	0.9	2.2	4.7
3	0	.6	2.2	4.6	7.4
4	.4	1.8	3.4	6.2	9.9
5	1.1	2.5	4.2	7.7	12.4
6	1.6	3.1	5.0	9.3	14.8
7	1.9	3.6	5.9	10.8	17.3
8	2.2	4.2	6.7	12.4	19.8
9	2.5	4.7	7.6	13.9	22.2
10	2.8	5.2	8.4	15.5	24.7
11	3.0	5.7	9.2	17.0	------
12	3.3	6.2	10.1	18.6	------
13	3.6	6.8	10.9	20.1	------
14	3.8	7.3	11.8	21.7	------
15	4.1	7.8	12.6	23.2	------
16	4.4	8.3	13.5	------	------
17	4.7	8.8	14.3	------	------
18	4.9	9.3	15.1	------	------
19	5.2	9.8	16.0	------	------
20	5.5	10.4	16.8	------	------

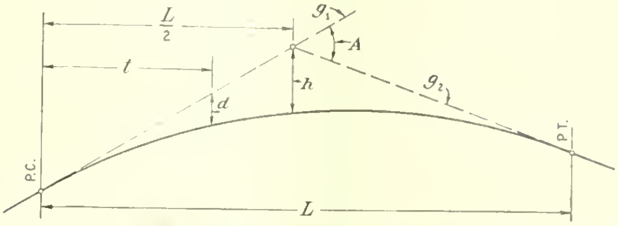


FIGURE 1.—CALCULATION OF ELEVATIONS ON VERTICAL CURVES.

TABLE 2.—*Minimum length of vertical curves to provide minimum passing sight distance*

A=algebraic difference of grades—percent	Length of vertical curve on 2-lane roads for assumed design speed of—					Length of vertical curve on 3-lane roads for assumed design speed of—		
	30 mi. per hr.	40 mi. per hr.	50 mi. per hr.	60 mi. per hr.	70 mi. per hr.	50 mi. per hr.	60 mi. per hr.	70 mi. per hr.
	Stations	*Stations*	*Stations*	*Stations*	*Stations*	*Stations*	*Stations*	*Stations*
1.00	------	------	------	6.0	22.0	------	------	0
1.25	------	------	0	13.2	29.2	------	0	7.2
1.50	------	------	4.0	18.0	35.0	------	2.0	12.0
1.75	------	------	7.4	21.4	40.9		5.4	15.4
2.00	------	0	10.0	24.5	46.7	0	8.0	18.0
2.5	------	3.6	13.6	30.6	------	3.6	11.6	22.5
3.0	------	6.0	16.3	36.8	------	6.0	14.1	27.0
3.5	0	7.7	19.1	42.9	------	7.7	16.4	31.5
4.0	1.0	9.0	21.8	49.0	------	9.0	18.8	36.0
4.5	2.0	10.1	24.5	------	------	10.1	21.1	40.5
5.0	2.8	11.2	27.2	------	------	11.2	23.5	45.0
5.5	3.5	12.4	29.9	------	------	12.4	25.8	------
6.0	4.0	13.5	32.7	------	------	13.5	28.2	------
7	4.9	15.8	38.1	------	------	15.8	32.9	------
8	5.6	18.0	43.6	------	------	18.0	37.6	------
9	6.2	20.2	49.0	------	------	20.2	42.2	------
10	6.9	22.5	------	------	------	22.5	46.9	------
11	7.6	24.8	------	------	------	24.8	------	------
12	8.3	27.0	------	------	------	27.0		------
13	9.0	29.2	------	------		29.2		------
14	9.7	31.5	------			31.5		
15	10.4	33.8	------	------		33.8		------
16	11.1	36.0	------	------		36.0		------

Calculations of elevations on vertical curves. Terms used are as follows (see fig. 1):

g_1 and g_2=grades, percent;
A=algebraic difference of grades, percent;
L=length of vertical curve, stations;
t=distance from end of curve, feet; and
d=vertical distance between grade line and vertical curve, feet.

The basic formulas are:

$$g_1-g_2=A$$

$$h=\frac{AL}{8}$$

$$\frac{d}{h}=\frac{t^2}{2{,}500\ L^2}$$

The value of d may be expressed as

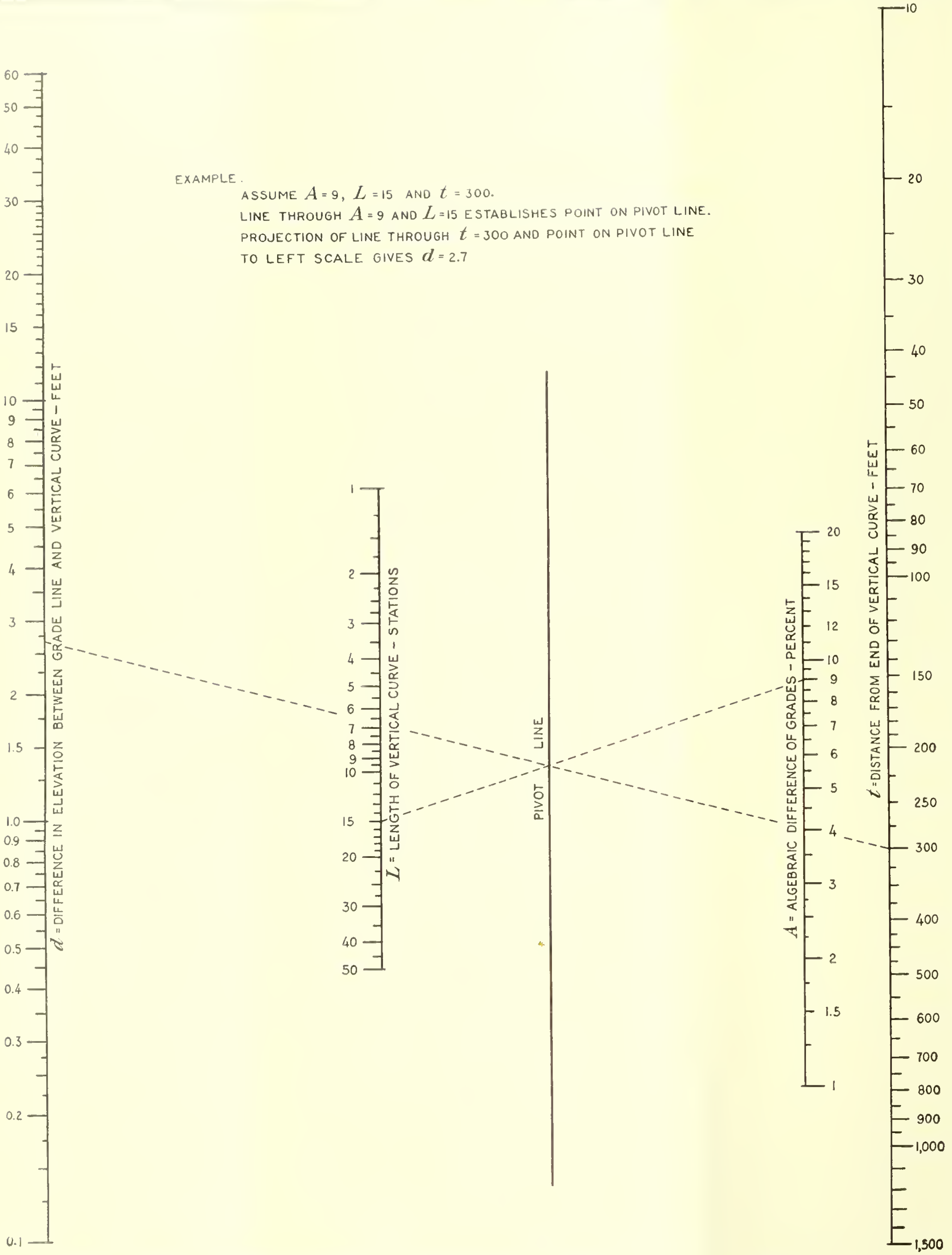
$$d=\frac{A}{L}\times K,$$

where

$$K=\frac{t^2}{20{,}000}.$$

Table 3 gives values of $\frac{A}{L}$. Values given are for whole numbers of A and L; for intermediate values interpolate or calculate $\frac{A}{L}$ directly. Table 4 gives values of K for various values of t. Then d= value of $\frac{A}{L}$ from table 3 times value of K from table 4.

Figure 2 is a nomograph which permits a more rapid but less accurate solution for one of the above factors when the other three are known.



EXAMPLE.

ASSUME $A = 9$, $L = 15$ AND $t = 300$.
LINE THROUGH $A = 9$ AND $L = 15$ ESTABLISHES POINT ON PIVOT LINE.
PROJECTION OF LINE THROUGH $t = 300$ AND POINT ON PIVOT LINE
TO LEFT SCALE GIVES $d = 2.7$

d = DIFFERENCE IN ELEVATION BETWEEN GRADE LINE AND VERTICAL CURVE - FEET
60
50
40
30
20
15
10
9
8
7
6
5
4
3
2
1.5
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
L = LENGTH OF VERTICAL CURVE - STATIONS
1
2
3
4
5
6
7
8
9
10
15
20
30
40
50
PIVOT LINE
A = ALGEBRAIC DIFFERENCE OF GRADES - PERCENT
20
15
12
10
9
8
7
6
5
4
3
2
1.5
1
t = DISTANCE FROM END OF VERTICAL CURVE - FEET
10
20
30
40
50
60
70
80
90
100
150
200
250
300
400
500
600
700
800
900
1,000
1,500

FIGURE 2.—NOMOGRAPH FOR CALCULATION OF ELEVATION OF A POINT ON A PARABOLIC VERTICAL CURVE.

TABLE 3.— *Values of* $\frac{A}{L}$, *for values of A from 1 to 16 and values of L from 1 to 30*

L (stations)	A=1 percent	A=2 percent	A=3 percent	A=4 percent	A=5 percent	A=6 percent	A=7 percent	A=8 percent	A=9 percent	A=10 percent	A=11 percent	A=12 percent	A=13 percent	A=14 percent	A=15 percent	A=16 percent
1	1.0000	2.0000	3.0000	4.0000	5.0000	6.0000	7.0000	8.0000	9.0000	10.0000	11.0000	12.0000	13.0000	14.0000	15.0000	16.0000
2	.5000	1.0000	1.5000	2.0000	2.5000	3.0000	3.5000	4.0000	4.5000	5.0000	5.5000	6.0000	6.5000	7.0000	7.5000	8.0000
3	.3333	.6667	1.0000	1.3333	1.6667	2.0000	2.3333	2.6667	3.0000	3.3333	3.6667	4.0000	4.3333	4.6667	5.0000	5.3333
4	.2500	.5000	.7500	1.0000	1.2500	1.5000	1.7500	2.0000	2.2500	2.5000	2.7500	3.0000	3.2500	3.5000	3.7500	4.0000
5	.2000	.4000	.6000	.8000	1.0000	1.2000	1.4000	1.6000	1.8000	2.0000	2.2000	2.4000	2.6000	2.8000	3.0000	3.2000
6	.1667	.3333	.5000	.6667	.8333	1.0000	1.1667	1.3333	1.5000	1.6667	1.8333	2.0000	2.1667	2.3333	2.5000	2.6667
7	.1428	.2857	.4286	.5714	.7143	.8571	1.0000	1.1428	1.2857	1.4286	1.5714	1.7143	1.8571	2.0000	2.1428	2.2857
8	.1250	.2500	.3750	.5000	.6250	.7500	.8750	1.0000	1.1250	1.2500	1.3750	1.5000	1.6250	1.7500	1.8750	2.0000
9	.1111	.2222	.3333	.4444	.5556	.6667	.7778	.8889	1.0000	1.1111	1.2222	1.3333	1.4444	1.5556	1.6667	1.7778
10	.1000	.2000	.3000	.4000	.5000	.6000	.7000	.8000	.9000	1.0000	1.1000	1.2000	1.3000	1.4000	1.5000	1.6000
11	.0909	.1818	.2727	.3636	.4545	.5454	.6364	.7273	.8182	.9091	1.0000	1.0909	1.1818	1.2727	1.3636	1.4545
12	.0833	.1667	.2500	.3333	.4167	.5000	.5833	.6667	.7500	.8333	.9167	1.0000	1.0833	1.1667	1.2500	1.3333
13	.0769	.1538	.2308	.3077	.3846	.4615	.5385	.6154	.6923	.7692	.8462	.9231	1.0000	1.0769	1.1538	1.2308
14	.0714	.1428	.2143	.2857	.3571	.4286	.5000	.5714	.6428	.7143	.7857	.8571	.9286	1.0000	1.0714	1.1428
15	.0667	.1333	.2000	.2667	.3333	.4000	.4667	.5333	.6000	.6667	.7333	.8000	.8667	.9333	1.0000	1.0667
16	.0625	.1250	.1875	.2500	.3125	.3750	.4375	.5000	.5625	.6250	.6875	.7500	.8125	.8750	.9375	1.0000
17	.0588	.1176	.1765	.2353	.2941	.3529	.4118	.4706	.5294	.5882	.6470	.7059	.7647	.8235	.8823	.9412
18	.0556	.1111	.1667	.2222	.2778	.3333	.3889	.4444	.5000	.5556	.6111	.6667	.7222	.7778	.8333	.8889
19	.0526	.1053	.1579	.2105	.2632	.3158	.3684	.4210	.4737	.5263	.5789	.6316	.6842	.7368	.7894	.8421
20	.0500	.1000	.1500	.2000	.2500	.3000	.3500	.4000	.4500	.5000	.5500	.6000	.6500	.7000	.7500	.8000
21	.0476	.0952	.1428	.1905	.2381	.2857	.3333	.3809	.4286	.4762	.5238	.5714	.6190	.6667	.7143	.7619
22	.0454	.0909	.1364	.1818	.2273	.2727	.3182	.3636	.4091	.4545	.5000	.5454	.5909	.6364	.6818	.7273
23	.0434	.0870	.1304	.1739	.2174	.2609	.3043	.3478	.3913	.4348	.4783	.5217	.5652	.6087	.6522	.6956
24	.0416	.0833	.1250	.1667	.2083	.2500	.2917	.3333	.3750	.4167	.4583	.5000	.5417	.5833	.6250	.6667
25	.0400	.0800	.1200	.1600	.2000	.2400	.2800	.3200	.3600	.4000	.4400	.4800	.5200	.5600	.6000	.6400
26	.0385	.0769	.1154	.1538	.1923	.2308	.2692	.3077	.3462	.3846	.4231	.4615	.5000	.5385	.5769	.6154
27	.0370	.0740	.1111	.1481	.1852	.2222	.2593	.2963	.3333	.3704	.4074	.4444	.4815	.5185	.5555	.5926
28	.0357	.0714	.1071	.1428	.1786	.2143	.2500	.2857	.3214	.3571	.3928	.4286	.4643	.5000	.5357	.5714
29	.0345	.0690	.1034	.1379	.1724	.2069	.2414	.2759	.3103	.3448	.3793	.4138	.4483	.4828	.5172	.5517
30	.0333	.0667	.1000	.1333	.1667	.2000	.2333	.2667	.3000	.3333	.3667	.4000	.4333	.4667	.5000	.5333

TABLE 4.—*Values of K for values of t from 1 to 1,500*

t	K	t	K	t	K	t	K
1	0.000	51	0.130	101	0.510	151	1.140
2	.000	52	.135	102	.520	152	1.155
3	.000	53	.140	103	.530	153	1.170
4	.001	54	.146	104	.541	154	1.186
5	.001	55	.151	105	.551	155	1.201
6	.002	56	.157	106	.562	156	1.217
7	.002	57	.162	107	.572	157	1.232
8	.003	58	.168	108	.583	158	1.248
9	.004	59	.174	109	.594	159	1.264
10	.005	60	.180	110	.605	160	1.280
11	.006	61	.186	111	.616	161	1.296
12	.007	62	.192	112	.627	162	1.312
13	.008	63	.198	113	.638	163	1.328
14	.010	64	.205	114	.650	164	1.345
15	.011	65	.211	115	.661	165	1.361
16	.013	66	.218	116	.673	166	1.378
17	.014	67	.224	117	.684	167	1.394
18	.016	68	.231	118	.696	168	1.411
19	.018	69	.238	119	.708	169	1.428
20	.020	70	.245	120	.720	170	1.445
21	.022	71	.252	121	.732	171	1.462
22	.024	72	.259	122	.744	172	1.479
23	.026	73	.266	123	.756	173	1.496
24	.029	74	.274	124	.769	174	1.514
25	.031	75	.281	125	.781	175	1.531
26	.034	76	.289	126	.794	176	1.549
27	.036	77	.296	127	.806	177	1.566
28	.039	78	.304	128	.819	178	1.584
29	.042	79	.312	129	.832	179	1.602
30	.045	80	.320	130	.845	180	1.620
31	.048	81	.328	131	.858	181	1.638
32	.051	82	.336	132	.871	182	1.656
23	.054	83	.344	133	.884	183	1.674
34	.058	84	.353	134	.898	184	1.693
35	.061	85	.361	135	.911	185	1.711
36	.065	86	.370	136	.925	186	1.730
37	.068	87	.378	137	.938	187	1.748
38	.072	88	.387	138	.952	188	1.767
39	.076	89	.396	139	.966	189	1.786
40	.080	90	.405	140	.980	190	1.805
41	.084	91	.414	141	.994	191	1.824
42	.088	92	.423	142	1.008	192	1.843
43	.092	93	.432	143	1.022	193	1.862
44	.097	94	.442	144	1.037	194	1.882
45	.101	95	.451	145	1.051	195	1.901
46	.106	96	.461	146	1.066	196	1.921
47	.110	97	.470	147	1.080	197	1.940
48	.115	98	.480	148	1.095	198	1.960
49	.120	99	.490	149	1.110	199	1.980
50	.125	100	.500	150	1.125	200	2.000

TABLE 4.—*Values of K for values of t from 1 to 1,500*— Con.

t	K	t	K	t	K	t	K
201	2.020	251	3.150	301	4.530	351	6.160
202	2.040	252	3.175	302	4.560	352	6.195
203	2.060	253	3.200	303	4.590	353	6.230
204	2.081	254	3.226	304	4.621	354	6.266
205	2.101	255	3.251	305	4.651	355	6.301
206	2.122	256	3.277	306	4.682	356	6.337
207	2.142	257	3.302	307	4.712	357	6.372
208	2.163	258	3.328	308	4.743	358	6.408
209	2.184	259	3.354	309	4.774	359	6.444
210	2.205	260	3.380	310	4.805	360	6.480
211	2.226	261	3.406	311	4.836	361	6.516
212	2.247	262	3.432	312	4.867	362	6.552
213	2.268	263	3.458	313	4.898	363	6.588
214	2.290	264	3.485	314	4.930	364	6.625
215	2.311	265	3.511	315	4.961	365	6.661
216	2.333	266	3.538	316	4.993	366	6.698
217	2.354	267	3.564	317	5.024	367	6.734
218	2.376	268	3.591	318	5.056	368	6.771
219	2.398	269	3.618	319	5.088	369	6.808
220	2.420	270	3.645	320	5.120	370	6.845
221	2.442	271	3.672	321	5.152	371	6.882
222	2.464	272	3.699	322	5.184	372	6.919
223	2.486	273	3.726	323	5.216	373	6.956
224	2.509	274	3.754	324	5.249	374	6.994
225	2.531	275	3.781	325	5.281	375	7.031
226	2.554	276	3.809	326	5.314	376	7.069
227	2.576	277	3.836	327	5.346	377	7.106
228	2.599	278	3.864	328	5.379	378	7.144
229	2.622	279	3.892	329	5.412	379	7.182
230	2.645	280	3.920	330	5.445	380	7.220
231	2.668	281	3.948	331	5.478	381	7.258
232	2.691	282	3.976	332	5.511	382	7.296
233	2.714	283	4.004	333	5.544	383	7.334
234	2.738	284	4.033	334	5.578	384	7.373
235	2.761	285	4.061	335	5.611	385	7.411
236	2.785	286	4.090	336	5.645	386	7.450
237	2.808	287	4.118	337	5.678	387	7.488
238	2.832	288	4.147	338	5.712	388	7.527
239	2.856	289	4.176	339	5.746	389	7.566
240	2.880	290	4.205	340	5.780	390	7.605
241	2.904	291	4.234	341	5.814	391	7.644
242	2.928	292	4.263	342	5.848	392	7.683
243	2.952	293	4.292	343	5.882	393	7.722
244	2.977	294	4.322	344	5.917	394	7.762
245	3.001	295	4.351	345	5.951	395	7.801
246	3.026	296	4.381	346	5.986	396	7.841
247	3.050	297	4.410	347	6.020	397	7.880
248	3.075	298	4.440	348	6.055	398	7.920
249	3.100	299	4.470	349	6.090	399	7.960
250	3.125	300	4.500	350	6.125	400	8.000

TABLE 4.—*Values of K for values of t from 1 to 1,500*—Con.

t	*K*	*t*	*K*	*t*	*K*	*t*	*K*
401	8.040	501	12.550	601	18.060	701	24.570
402	8.080	502	12.600	602	18.120	702	24.640
403	8.120	503	12.650	603	18.180	703	24.710
404	8.161	504	12.701	604	18.241	704	24.781
405	8.201	505	12.751	605	18.301	705	24.851
406	8.242	506	12.802	606	18.362	706	24.922
407	8.282	507	12.852	607	18.422	707	24.992
408	8.323	508	12.903	608	18.483	708	25.063
409	8.364	509	12.954	609	18.544	709	25.134
410	8.405	510	13.005	610	18.605	710	25.205
411	8.446	511	13.056	611	18.666	711	25.276
412	8.487	512	13.107	612	18.727	712	25.347
413	8.528	513	13.158	613	18.788	713	25.418
414	8.570	514	13.210	614	18.850	714	25.490
415	8.611	515	13.261	615	18.911	715	25.561
416	8.653	516	13.313	616	18.973	716	25.633
417	8.694	517	13.364	617	19.034	717	25.704
418	8.736	518	13.416	618	19.096	718	25.776
419	8.778	519	13.468	619	19.158	719	25.848
420	8.820	520	13.520	620	19.220	720	25.920
421	8.862	521	13.572	621	19.282	721	25.992
422	8.904	522	13.624	622	19.344	722	26.064
423	8.946	523	13.676	623	19.406	723	26.136
424	8.989	524	13.729	624	19.469	724	26.209
425	9.031	525	13.781	625	19.531	725	26.281
426	9.074	526	13.834	626	19.594	726	26.354
427	9.116	527	13.886	627	19.656	727	26.426
428	9.159	528	13.939	628	19.719	728	26.499
429	9.202	529	13.992	629	19.782	729	26.572
430	9.245	530	14.045	630	19.845	730	26.645
431	9.288	531	14.098	631	19.908	731	26.718
432	9.331	532	14.151	632	19.971	732	26.791
433	9.374	533	14.204	633	20.034	733	26.864
434	9.418	534	14.258	634	20.098	734	26.938
435	9.461	535	14.311	635	20.161	735	27.011
436	9.505	536	14.365	636	20.225	736	27.085
437	9.548	537	14.418	637	20.288	737	27.158
438	9.592	538	14.472	638	20.352	738	27.232
439	9.636	539	14.526	639	20.416	739	27.306
440	9.680	540	14.580	640	20.480	740	27.380
441	9.724	541	14.634	641	20.544	741	27.454
442	9.768	542	14.688	642	20.608	742	27.528
443	9.812	543	14.742	643	20.672	743	27.602
444	9.857	544	14.797	644	20.737	744	27.677
445	9.901	545	14.851	645	20.801	745	27.751
446	9.946	546	14.906	646	20.866	746	27.826
447	9.990	547	14.960	647	20.930	747	27.900
448	10.035	548	15.015	648	20.995	748	27.975
449	10.080	549	15.070	649	21.060	749	28.050
450	10.125	550	15.125	650	21.125	750	28.125
451	10.170	551	15.180	651	21.190	751	28.200
452	10.215	552	15.235	652	21.255	752	28.275
453	10.260	553	15.290	653	21.320	753	28.350
454	10.306	554	15.346	654	21.386	754	28.426
455	10.351	555	15.401	655	21.451	755	28.501
456	10.397	556	15.457	656	21.517	756	28.577
457	10.442	557	15.512	657	21.582	757	28.652
458	10.488	558	15.568	658	21.648	758	28.728
459	10.534	559	15.624	659	21.714	759	28.804
460	10.580	560	15.680	660	21.780	760	28.880
461	10.626	561	15.736	661	21.846	761	28.956
462	10.672	562	15.792	662	21.912	762	29.032
463	10.718	563	15.848	663	21.978	763	29.108
464	10.765	564	15.905	664	22.045	764	29.185
465	10.811	565	15.961	665	22.111	765	29.261
466	10.858	566	16.018	666	22.178	766	29.338
467	10.904	567	16.074	667	22.244	767	29.414
468	10.951	568	16.131	668	22.311	768	29.491
469	10.998	569	16.188	669	22.378	769	29.568
470	11.045	570	16.245	670	22.445	770	29.645
471	11.092	571	16.302	671	22.512	771	29.722
472	11.139	572	16.359	672	22.579	772	29.799
473	11.186	573	16.416	673	22.646	773	29.876
474	11.234	574	16.474	674	22.714	774	29.954
475	11.281	575	16.531	675	22.781	775	30.031
476	11.329	576	16.589	676	22.849	776	30.109
477	11.376	577	16.646	677	22.916	777	30.186
478	11.424	578	16.704	678	22.984	778	30.264
479	11.472	579	16.762	679	23.052	779	30.342
480	11.520	580	16.820	680	23.120	780	30.420
481	11.568	581	16.878	681	23.188	781	30.498
482	11.616	582	16.936	682	23.256	782	30.576
483	11.664	583	16.994	683	23.324	783	30.654
484	11.713	584	17.053	684	23.393	784	30.733
485	11.761	585	17.111	685	23.461	785	30.811
486	11.810	586	17.170	686	23.530	786	30.890
487	11.858	587	17.228	687	23.598	787	30.968
488	11.907	588	17.287	688	23.667	788	31.047
489	11.956	589	17.346	689	23.736	789	31.126
490	12.005	590	17.405	690	23.805	790	31.205
491	12.054	591	17.464	691	23.874	791	31.284
492	12.103	592	17.523	692	23.943	792	31.363
493	12.152	593	17.582	693	24.012	793	31.442
494	12.202	594	17.642	694	24.082	794	31.522
495	12.251	595	17.701	695	24.151	795	31.601
496	12.301	596	17.761	696	24.221	796	31.681
497	12.350	597	17.820	697	24.290	797	31.760
498	12.400	598	17.880	698	24.360	798	31.840
499	12.450	599	17.940	699	24.430	799	31.920
500	12.500	600	18.000	700	24.500	800	32.000

TABLE 4.—*Values of K for values of t from 1 to 1,500*—Con.

t	*K*	*t*	*K*	*t*	*K*	*t*	*K*
801	32.080	901	40.590	1,001	50.100	1,101	60.610
802	32.160	902	40.680	1,002	50.200	1,102	60.720
803	32.240	903	40.770	1,003	50.300	1,103	60.830
804	32.321	904	40.861	1,004	50.401	1,104	60.941
805	32.401	905	40.951	1,005	50.501	1,105	61.051
806	32.482	906	41.042	1,006	50.602	1,106	61.162
807	32.562	907	41.132	1,007	50.702	1,107	61.272
808	32.643	908	41.223	1,008	50.803	1,108	61.383
809	32.724	909	31.314	1,009	50.904	1,109	61.494
810	32.805	910	41.405	1,010	51.005	1,110	61.605
811	32.886	911	41.496	1,011	51.106	1,111	61.716
812	32.967	912	41.587	1,012	51.207	1,112	61.827
813	33.048	913	41.678	1,013	51.308	1,113	61.938
814	33.130	914	41.770	1,014	51.409	1,114	62.050
815	33.211	915	41.861	1,015	51.511	1,115	62.161
816	33.293	916	41.953	1,016	51.613	1,116	62.273
817	33.374	917	42.044	1,017	51.714	1,117	62.384
818	33.456	918	42.136	1,018	51.816	1,118	62.496
819	33.538	919	42.228	1,019	51.918	1,119	62.608
820	33.620	920	42.320	1,020	52.020	1,120	62.720
821	33.702	921	42.412	1,021	52.122	1,121	62.832
822	33.784	922	42.504	1,022	52.224	1,122	62.944
823	33.866	923	42.596	1,023	52.326	1,123	63.056
824	33.949	924	42.689	1,024	52.428	1,124	63.169
825	34.031	925	42.781	1,025	52.531	1,125	63.281
826	34.114	926	42.874	1,026	52.634	1,126	63.394
827	34.196	927	42.966	1,027	52.736	1,127	63.506
828	34.279	928	43.059	1,028	52.839	1,128	63.619
829	34.362	929	43.152	1,029	52.942	1,129	63.732
830	34.445	930	43.245	1,030	53.045	1,130	63.845
831	34.528	931	43.338	1,031	53.148	1,131	63.958
832	34.611	932	43.431	1,032	53.251	1,132	64.071
833	34.694	933	43.524	1,033	53.354	1,133	64.184
834	34.778	934	43.618	1,034	53.458	1,134	64.298
835	34.861	935	43.711	1,035	53.561	1,135	64.411
836	34.945	936	43.805	1,036	53.665	1,136	64.525
837	35.028	937	43.898	1,037	53.768	1,137	64.638
838	35.112	938	43.992	1,038	53.872	1,138	64.752
839	35.196	939	44.086	1,039	53.976	1,139	64.866
840	35.280	940	44.180	1,040	54.080	1,140	64.980
841	35.364	941	44.274	1,041	54.184	1,141	65.094
842	35.448	942	44.368	1,042	54.288	1,142	65.208
843	35.532	943	44.462	1,043	54.392	1,143	65.322
844	35.617	944	44.557	1,044	54.497	1,144	65.437
845	35.701	945	44.651	1,045	54.601	1,145	65.551
846	35.786	946	44.746	1,046	54.706	1,146	65.666
847	35.870	947	44.840	1,047	54.810	1,147	65.780
848	35.955	948	44.935	1,048	54.915	1,148	65.895
849	36.040	949	45.030	1,049	55.020	1,149	66.010
850	36.125	950	45.125	1,050	55.125	1,150	66.125
851	36.210	951	45.220	1,051	55.230	1,151	66.240
852	36.295	952	45.315	1,052	55.335	1,152	66.355
853	36.380	953	45.410	1,053	55.440	1,153	66.470
854	36.466	954	45.506	1,054	55.546	1,154	66.586
855	36.551	955	45.601	1,055	55.651	1,155	66.701
856	36.637	956	45.697	1,056	55.757	1,156	66.817
857	36.722	957	45.742	1,057	55.862	1,157	66.932
858	36.808	958	45.888	1,058	55.968	1,158	67.048
859	36.894	959	45.984	1,059	56.074	1,159	67.164
860	36.980	960	46.080	1,060	56.180	1,160	67.280
861	37.066	961	46.176	1,061	56.286	1,161	67.396
862	37.152	962	46.272	1,062	56.392	1,162	67.512
863	37.238	963	46.368	1,063	56.498	1,163	67.628
864	37.325	964	46.465	1,064	56.605	1,164	67.745
865	37.411	965	46.561	1,065	56.711	1,165	67.861
866	37.498	966	46.657	1,066	56.818	1,166	67.978
867	37.584	967	46.754	1,067	56.924	1,167	68.094
868	37.671	968	46.851	1,068	57.031	1,168	68.211
869	37.758	969	46.948	1,069	57.138	1,169	68.328
870	37.845	970	47.045	1,070	57.245	1,170	68.445
871	37.932	971	47.142	1,071	57.352	1,171	68.562
872	38.019	972	47.239	1,072	57.459	1,172	68.679
873	38.106	973	47.336	1,073	57.566	1,173	68.796
874	38.194	974	47.434	1,074	57.674	1,174	68.914
875	38.281	975	47.531	1,075	57.781	1,175	69.031
876	38.369	976	47.629	1,076	57.889	1,176	69.149
877	38.456	977	47.726	1,077	57.996	1,177	69.266
878	38.544	978	47.824	1,078	58.104	1,178	69.384
879	38.632	979	47.922	1,079	58.212	1,179	69.502
880	38.720	980	48.020	1,080	58.320	1,180	69.620
881	38.808	981	48.118	1,081	58.428	1,181	69.738
882	38.896	982	48.216	1,082	58.536	1,182	69.856
883	38.984	983	48.314	1,083	58.644	1,183	69.974
884	39.073	984	48.413	1,084	58.753	1,184	70.093
885	39.161	985	48.511	1,085	58.861	1,185	70.211
886	39.250	986	48.610	1,086	58.970	1,186	70.330
887	39.338	987	48.708	1,087	59.078	1,187	70.448
888	39.427	988	48.807	1,088	59.187	1,188	70.567
889	39.516	989	48.906	1,089	59.296	1,189	70.686
890	39.605	990	49.005	1,090	59.405	1,190	70.805
891	39.694	991	49.104	1,091	59.514	1,191	70.924
892	39.783	992	49.203	1,092	59.623	1,192	71.043
893	39.872	993	49.302	1,093	59.732	1,193	71.162
894	39.962	994	49.402	1,094	59.842	1,194	71.282
895	40.051	995	49.501	1,095	59.951	1,195	71.401
896	40.141	996	49.601	1,096	60.061	1,196	71.521
897	40.230	997	49.700	1,097	60.170	1,197	71.640
898	40.320	998	49.800	1,098	60.280	1,198	71.760
899	40.410	999	49.900	1,099	60.390	1,199	71.880
900	40.500	1,000	50.000	1,100	60.500	1,200	72.000

TABLE 4.—*Values of K for values of t from 1 to 1,500*—Con.

t	*K*	*t*	*K*	*t*	*K*	*t*	*K*
1,201	72.120	1,246	77.626	1,291	83.334	1,336	89.245
1,202	72.240	1,247	77.750	1,292	83.463	1,337	89.378
1,203	72.360	1,248	77.875	1,293	83.592	1,338	89.512
1,204	72.481	1,249	78.000	1,294	83.722	1,339	89.646
1,205	72.601	1,250	78.125	1,295	83.851	1,340	89.780
1,206	72.722	1,251	78.250	1,296	83.981	1,341	89.914
1,207	72.842	1,252	78.375	1,297	84.110	1,342	90.048
1,208	72.963	1,253	78.500	1,298	84.240	1,343	90.182
1,209	73.084	1,254	78.626	1,299	84.370	1,344	90.317
1,210	73.205	1,255	78.751	1,300	84.500	1,345	90.451
1,211	73.326	1,256	78.877	1,301	84.630	1,346	90.586
1,212	73.447	1,257	79.002	1,302	84.760	1,347	90.720
1,213	73.568	1,258	79.128	1,303	84.890	1,348	90.855
1,214	73.690	1,259	79.254	1,304	85.021	1,349	90.990
1,215	73.811	1,260	79.380	1,305	85.151	1,350	91.125
1,216	73.933	1,261	79.506	1,306	85.282	1,351	91.260
1,217	74.054	1,262	79.632	1,307	85.412	1,352	91.395
1,218	74,176	1,263	79.758	1,308	85.543	1,353	91.530
1,219	74.298	1,264	79.885	1,309	85.674	1,354	91.666
1,220	74.420	1,265	80.011	1,310	85.805	1,355	91.801
1,221	74.542	1,266	80.138	1,311	85.936	1,356	91.937
1,222	74.664	1,267	80.264	1,312	86.067	1,357	92.072
1,223	74.786	1,268	80.391	1,313	86.198	1,358	92.208
1,224	74.909	1,269	80.518	1,314	86.330	1,359	92.344
1,225	75.031	1,270	80.645	1,315	86.461	1,360	92.480
1,226	75.154	1,271	80.772	1,316	86.593	1,361	92.616
1,227	75.276	1,272	80.899	1,317	86.724	1,362	92.752
1,228	75.399	1,273	81.026	1,318	86.856	1,363	92.888
1,229	75.522	1,274	81.154	1,319	86.988	1,364	93.025
1,230	75.645	1,275	81.281	1,320	87.120	1,365	93.161
1,231	75.768	1,276	81.409	1,321	87.252	1,366	93.298
1,232	75.891	1,277	81.536	1,322	87.384	1,367	93.434
1,233	76.014	1,278	81.664	1,323	87.516	1,368	93.571
1,234	76.138	1,279	81.792	1,324	87.649	1,369	93.708
1,235	76.261	1,280	81.920	1,325	87.781	1,370	93.845
1,236	76.385	1,281	82.048	1,326	87.914	1,371	93.982
1,237	76.508	1,282	82.176	1,327	88.046	1,372	94.119
1,238	76.632	1,283	82.304	1,328	88.179	1,373	94.256
1,239	76.756	1,284	82.433	1,329	88.312	1,374	94.393
1,240	76.880	1,285	82.561	1,330	88.445	1,375	94.531
1,241	77.004	1,286	82.690	1,331	88.578	1,376	94.669
1,242	77.128	1,287	82.818	1,332	88.711	1,377	94.806
1,243	77.252	1,288	82.947	1,333	88.844	1,378	94.944
1,244	77.377	1,289	83.076	1,334	88.978	1,379	95.082
1,245	77.501	1,290	83.205	1,335	89.111	1,380	95.220

TABLE 4.—*Values of K for values of t from 1 to 1,500*—Con.

t	K	t	K	t	K	t	K
1,381	95.358	1,411	99.546	1,441	103.824	1,471	108.192
1,382	95.496	1,412	99.687	1,442	103.968	1,472	108.339
1,383	95.634	1,413	99.828	1,443	104.112	1,473	108.486
1,384	95.773	1,414	99.970	1,444	104.257	1,474	108.634
1,385	95.911	1,415	100.111	1,445	104.401	1,475	108.781
1,386	96.050	1,416	100.253	1,446	104.546	1,476	108.929
1,387	96.188	1,417	100.394	1,447	104.690	1,477	109.076
1,388	96.327	1,418	100.536	1,448	104.835	1,478	109.224
1,389	96.466	1,419	100.678	1,449	104.980	1,479	109.372
1,390	96.605	1,420	100.820	1,450	105.125	1,480	109.520
1,391	96.744	1,421	100.962	1,451	105.270	1,481	109.668
1,392	96.883	1,422	101.104	1,452	105.415	1,482	109.816
1,393	97.022	1,423	101.246	1,453	105.560	1,483	109.964
1,394	97.162	1,424	101.389	1,454	105.706	1,484	110.113
1,395	97.301	1,425	101.531	1,455	105.851	1,485	110.261
1,396	97.440	1,426	101.674	1,456	105.997	1,486	110.410
1,397	97.580	1,427	101.816	1,457	106.142	1,487	110.558
1,398	97.720	1,428	101.959	1,458	106.288	1,488	110.707
1,399	97.860	1,429	102.102	1,459	106.434	1,489	110.856
1,400	98.000	1,430	102.245	1,460	106.580	1,490	111.005
1,401	98.140	1,431	102.388	1,461	106.726	1,491	111.154
1,402	98.280	1,432	102.531	1,462	106.872	1,492	111.303
1,403	98.420	1,433	102.674	1,463	107.018	1,493	111.452
1,404	98.561	1,434	102.818	1,464	107.165	1,494	111.602
1,405	98.701	1,435	102.961	1,465	107.311	1,495	111.751
1,406	98.842	1,436	103.105	1,466	107.458	1,496	111.901
1,407	98.982	1,437	103.248	1,467	107.604	1,497	112.050
1,408	99.123	1,438	103.392	1,468	107.751	1,498	112.200
1,409	99.264	1,439	103.536	1,469	107.898	1,499	112.350
1,410	99.405	1,440	103.680	1,470	108.045	1,500	112.500

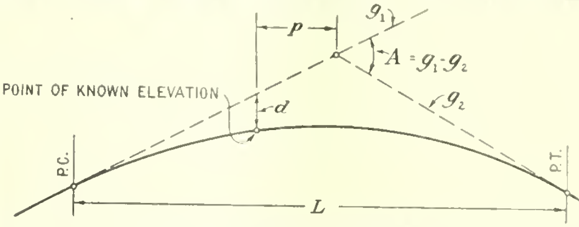
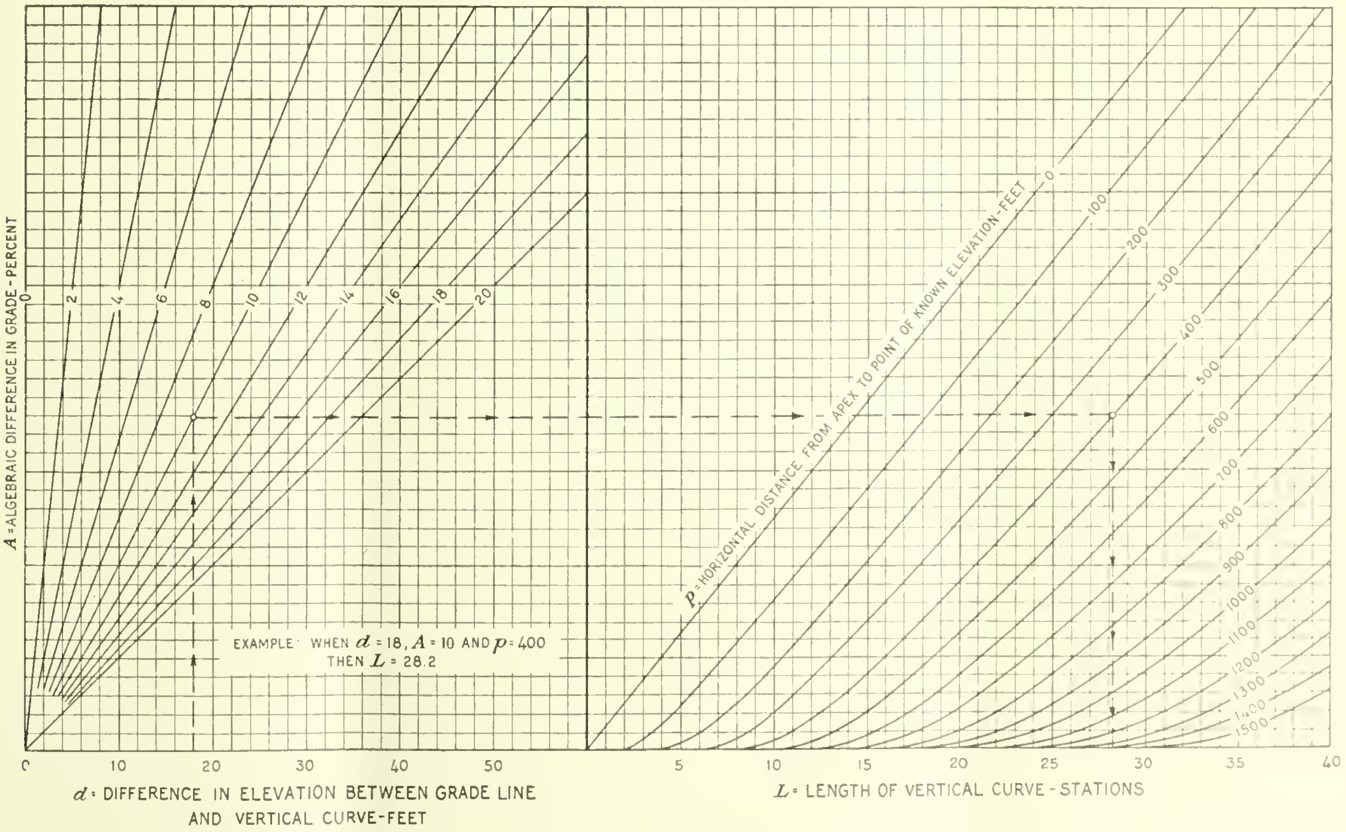
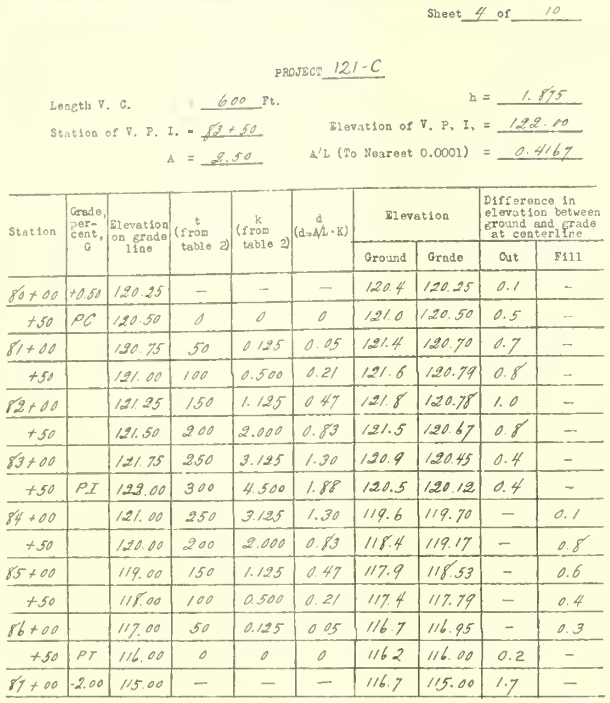


FIGURE 3.—LENGTH OF VERTICAL CURVE TO PASS THROUGH A POINT OF KNOWN ELEVATION.



EXAMPLE: WHEN $d = 18$, $A = 10$ AND $p = 400$ THEN $L = 28.2$

FIGURE 4.—GRAPHICAL SOLUTION FOR LENGTH OF VERTICAL CURVE TO PASS THROUGH A POINT OF KNOWN ELEVATION.



Sheet 4 of 10

PROJECT 121-C

Length V. C. 600 Ft.
h = 1.875
Station of V. P. I. = 83+50
Elevation of V. P. I. = 122.00
A = 2.50
A/L (To Nearest 0.0001) = 0.4167

Station	Grade, per-cent, G	Elevation on grade line	t (from table 2)	k (from table 2)	d (d=A/L·K)	Elevation		Difference in elevation between ground and grade at centerline	
						Ground	Grade	Cut	Fill
80+00	+0.50	120.25	—	—	—	120.4	120.25	0.1	–
+50	PC	120.50	0	0	0	121.0	120.50	0.5	–
81+00		120.75	50	0 125	0.05	121.4	120.70	0.7	–
+50		121.00	100	0.500	0.21	121.6	120.79	0.8	–
82+00		121.25	150	1.125	0 47	121.8	120.78	1.0	–
+50		121.50	200	2.000	0.83	121.5	120.67	0.8	–
83+00		121.75	250	3.125	1.30	120.9	120.45	0.4	–
+50	PI	122.00	300	4.500	1.88	120.5	120.12	0.4	–
84+00		121.00	250	3.125	1.30	119.6	119.70	—	0.1
+50		120.00	200	2.000	0.83	118.4	119.17	—	0.8
85+00		119.00	150	1.125	0.47	117.9	118.53	–	0.6
+50		118.00	100	0.500	0.21	117.4	117.79	—	0.4
86+00		117.00	50	0.125	0 05	116.7	116.95	–	0.3
+50	PT	116.00	0	0	0	116.2	116.00	0.2	–
87+00	-2.00	115.00	—	—	—	116.7	115.00	1.7	—

FIGURE 5. SAMPLE SHEET FOR CALCULATION OF A VERTICAL CURVE.

Length of vertical curve to pass through a point of known elevation. Values of the following are known (see fig. 3):

A=algebraic difference of grades, percent;
p=horizontal distance from apex to point of known elevation, feet; and
d=difference in elevation between grade line and vertical curve at point of known elevation, feet.

It is desired to determine L, the length of vertical curve (in stations) to pass through point of known elevation. The following basic formula is used:

$$d=\frac{A\ (50L-p)^2}{20{,}000\ L}$$

A direct graphical solution for value of L is given in figure 4. The steps in using this figure are:

1. Enter at value of d (lower left scale) and project upward to value of A (left sloping lines).
2. Project horizontally across to value of p (right sloping lines).
3. Project down to read value of L (lower right scale).

Figure 4 may also be used to solve for value of d when values of A, L, and p are known.

Figure 5 is a sample sheet for calculation of a vertical curve using the values of tables 3 and 4.

COTTON FABRIC FOR EROSION CONTROL

REPORT OF EXPERIMENTS ON ROADSIDE SLOPES AND DITCHES IN MISSISSIPPI

BY DISTRICT 8, PUBLIC ROADS ADMINISTRATION

Reported by W. B. KING, Associate Bridge Construction Engineer

COTTON FABRIC was used experimentally for erosion control on roadside slopes and ditches on eight test sections in Mississippi during the summer and fall of 1940. The fabric was furnished by the Division of Marketing and Marketing Agreements of the United States Department of Agriculture, and was installed by the Mississippi State Highway Department.

The fabric used was of the following types: S1–54, S1–40, S2–54, and S2–40, the latter figure in each designation being the width of the fabric in inches. Approximately equal amounts of each type of fabric were used, 15,948 square yards being used in the eight test sections. Both types of cotton fabric have relatively large mesh openings, type S1 having 6 openings per inch and type S2 having 4. Table 1 shows the amount of each type of fabric and use on each test section.

The fabric was delivered in rolls. On some areas of the test sections the fabric was run horizontally; on others it was run from top to toe of the slope, or "vertically."

Descriptions of each test section and the results obtained follow:

Section A.—The soil was sand-clay, clay, and marl. The areas were first treated by spreading a mixture of bermuda grass roots and topsoil to a depth of approximately 2 inches. The grass mulch was spread uniformly with shovels and rakes. The fabric was then applied and pegged down with galvanized wire staples. After passing through one growing season there appeared to be little difference in the amount of growth on the covered and uncovered areas. The fabric-covered slope was greener and the ground somewhat more moist than was the uncovered slope. The fabric in the ditch bottom appeared definitely to have retarded erosion.

Figure 1 shows the appearance of portions of Section A soon after the fabric was placed and the appearance of the same portions after one growing season.

TABLE 1.—*Amount of each type of cotton fabric and use on each test section*

Section No.	Project No.	Amount of cotton fabric, type—		Used on—
		S1	S2	
		Sq. yd.	*Sq. yd.*	
A	PWS 72-C	1,305	750	3:1 to 6:1 slopes; ditch bottom.
B	PWS 123	1,500	2,250	2:1 backslopes.
C	PWS 82-C	749	999	2½:1 and 4:1 backslopes; ditch bottom.
D	NRH 74-G	1,361	109	2½:1 backslopes; ditch bottom.
E	PWA 254-A	1,200	1,215	3:1 backslope; ditch bottom.
F	FAP 173-A	1,100	1,105	1½:1 and 2:1 backslopes.
G	NRS 236	555	750	2:1 backslopes.
H	FAP 45	500	500	2:1 backslopes.
Total		8,270	7,678	

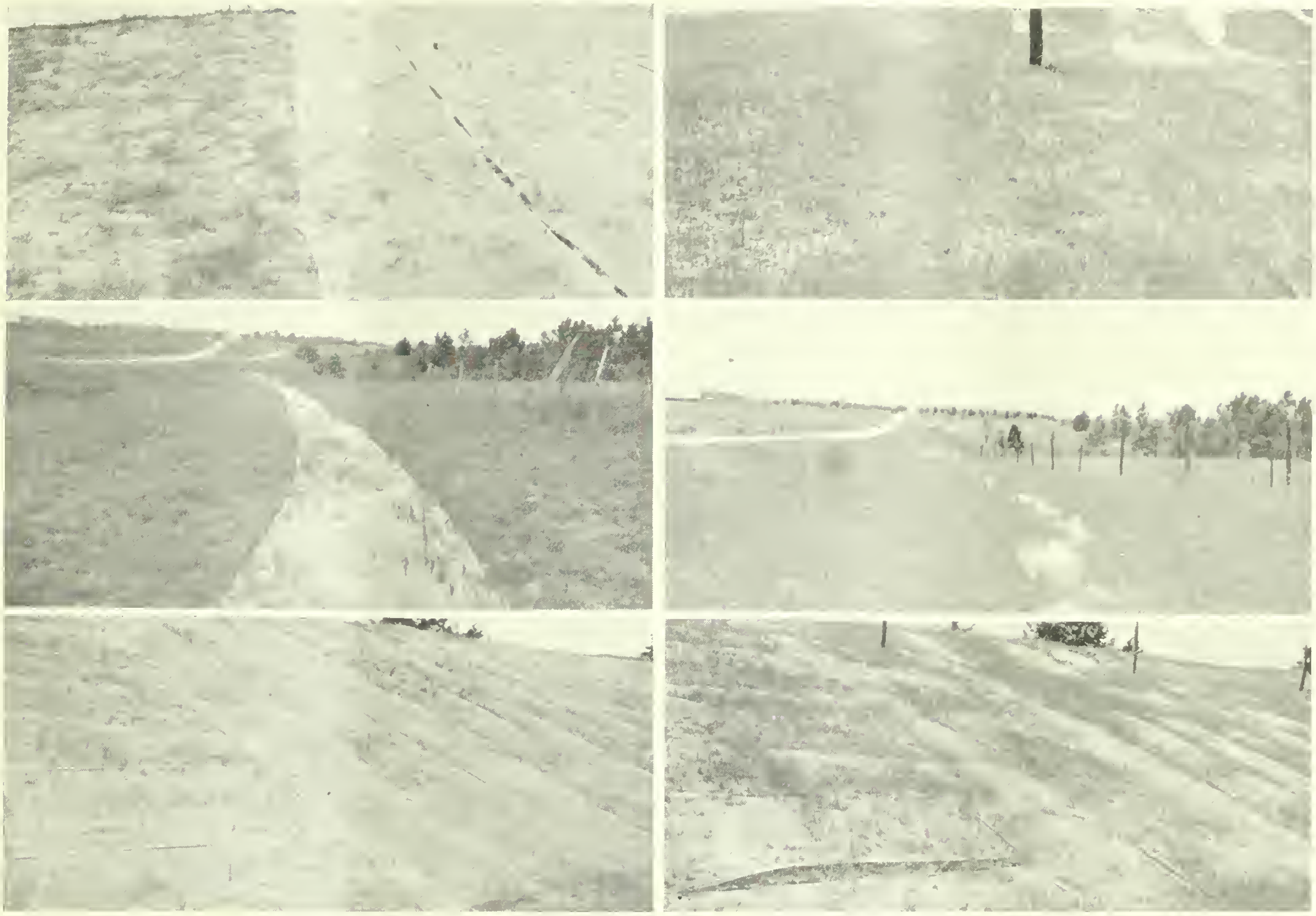


Figure 1.—Left, Appearance of Portions of Section A Soon After Fabric Was Placed; Right, Appearance of the Same Respective Portions of Section A After One Growing Season.

Section B.—The soil was sand, sand-clay, and clay. The backslopes were treated the same as the above section; and in addition commercial 4–8–4 fertilizer was mixed into the topsoil at the rate of 750 pounds per acre, and bermuda and carpet grass seed were planted. Part of the treated area was covered with cotton fabric pegged down with wooden stakes, and part was covered with straw mulch. After one growing season the fabric appeared to have helped the growth by shading and retarding erosion. However, the straw mulch appeared to have given better results more cheaply.

Section C.—The soil was topsoil, clay, sand-clay, and rock. The 2½:1 slopes and the ditch bottom were treated with topsoil and grass roots the same as on Section A. The 4:1 slope, consisting of clay and sand-clay soil, received no treatment except the cotton fabric covering. The fabric was pegged down with galvanized wire staples. After one growing season, the fabric in the ditch bottom appeared definitely to have retarded erosion and aided the growth. The fabric on the 2½:1 slopes helped by shading the growth. No difference was noted in the amount of erosion on the covered and uncovered portions. On the 4:1 slope, where no grass or topsoil was provided, the fabric appeared definitely to have retarded erosion and promoted native growth.

Section D.—The soil was a mixture of sand and clay. The areas were treated with topsoil and grass roots the same as on Section A, and the fabric similarly pegged with galvanized wire staples. At the time of inspection after one growing season, a large portion of the fabric on the slopes had disappeared. No appreciable difference could be noted in the portion of the slopes that was originally covered with fabric and the portion that was left uncovered. The fabric in the ditch bottom appeared to have helped retard erosion.

Figure 2 shows the appearance of portions of Sections B, C, and D after the fabric was placed, and the appearance of the same sections after one growing season.

Section E.—The soil was a sandy clay. A short time prior to placing the fabric the slopes were plowed, fertilized, seeded with grass, and sprigged. The grass had attained a fair growth prior to applying the fabric. A short time after the fabric was placed, two-thirds of it was destroyed by fire. No appreciable difference was noted between areas where the bank remained covered and where it was uncovered. Some erosion was noted between the grass rows beneath the fabric that remained in place. The fabric helped to retard erosion in the ditch bottom.

Section F.—The soil was limestone, hard-pan, sand-clay, and clay. The fabric was placed on backslopes that had been strip-sodded about 2 years previously. Some fabric was used on backslopes without sod. The fabric was pegged down with wooden stakes. Nine months after installation an inspection revealed that on the upper part of the north bank growth had benefited

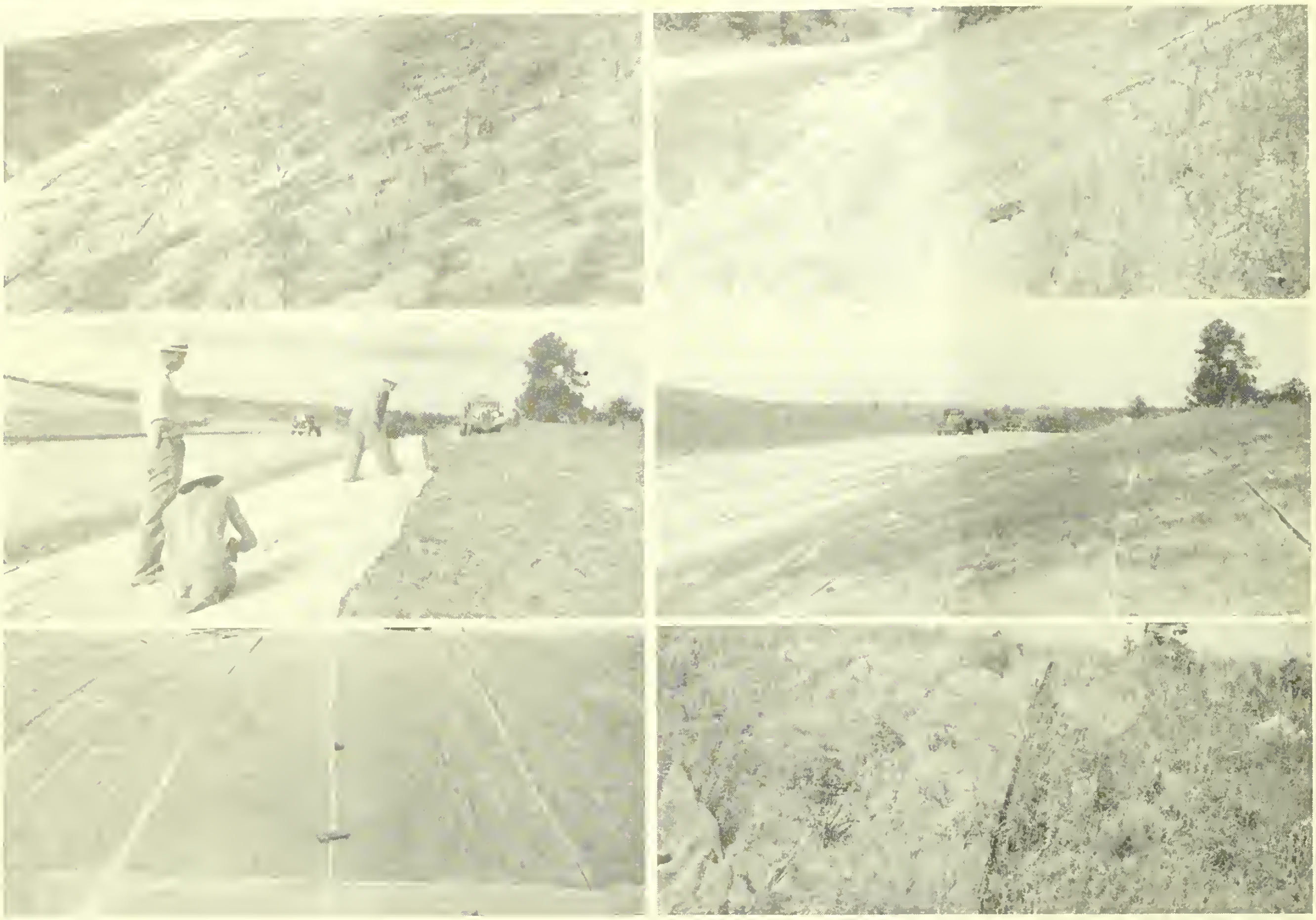


Figure 2.—Left, Appearance of Portions of Sections B (Top), C (Middle), and D (Bottom) During or Soon After Placing of Fabric; Right, Appearance of the Same Respective Portions After One Growing Season.

by the shade afforded by the fabric. No benefit was noted on the limestone and clay soil at the bottom of the north bank. On the south bank the growth under the fabric appeared to have benefited by the shade, it being greener than the uncovered portions. No other difference between the covered and uncovered portions was noted.

Section G.—The slopes had been covered with 4 inches of good topsoil containing bermuda and other native grasses. The grass had attained a fair growth before the fabric was placed. However, the results could not be compared with those for other sections because on two of the banks the fabric had been covered with straw mulch and on the third bank the fabric had been removed and the bank covered with straw mulch.

Section H.—The soil in the slopes was sand-clay. All of the fabric was placed on sodded slopes where the grass was already well established. No results could be obtained because all of the fabric was torn loose by cattle and hogs a short time after it was placed.

The results obtained on these eight experimental sections indicate that:

1. There is no apparent advantage in using cotton fabric on 3:1 or flatter slopes that have been treated with a mixture of grass roots and topsoil.

2. Use of cotton fabric has some advantage in shading growth and retarding erosion on 2:1 and 1½:1 slopes that have been treated with a mixture of grass roots and topsoil, or sprigged, or strip sodded. However, straw mulch serves better and is more economical wherever available.

3. Cotton fabric aids natural growth and retards erosion on 4:1 slopes of sandy clay and clay soils that have received no other treatment. However, straw mulch serves better wherever available.

4. There is definite advantage in using fabric in roadside ditch bottoms in promoting growth and retarding erosion. The fabric should be pegged at short intervals across its width to hold it close to the ground.

5. The S2–54 fabric is more effective than the other types. The open mesh of the S2 fabric allows vegetation to grow through it, and the wider material is more economical to install per square yard.

6. For greatest effectiveness, the fabric should be placed up and down on slopes and parallel to the flow line in ditches.

7. Galvanized wire staples are more effective than wooden stakes in pegging the fabric.

(*Continued from page 88*)

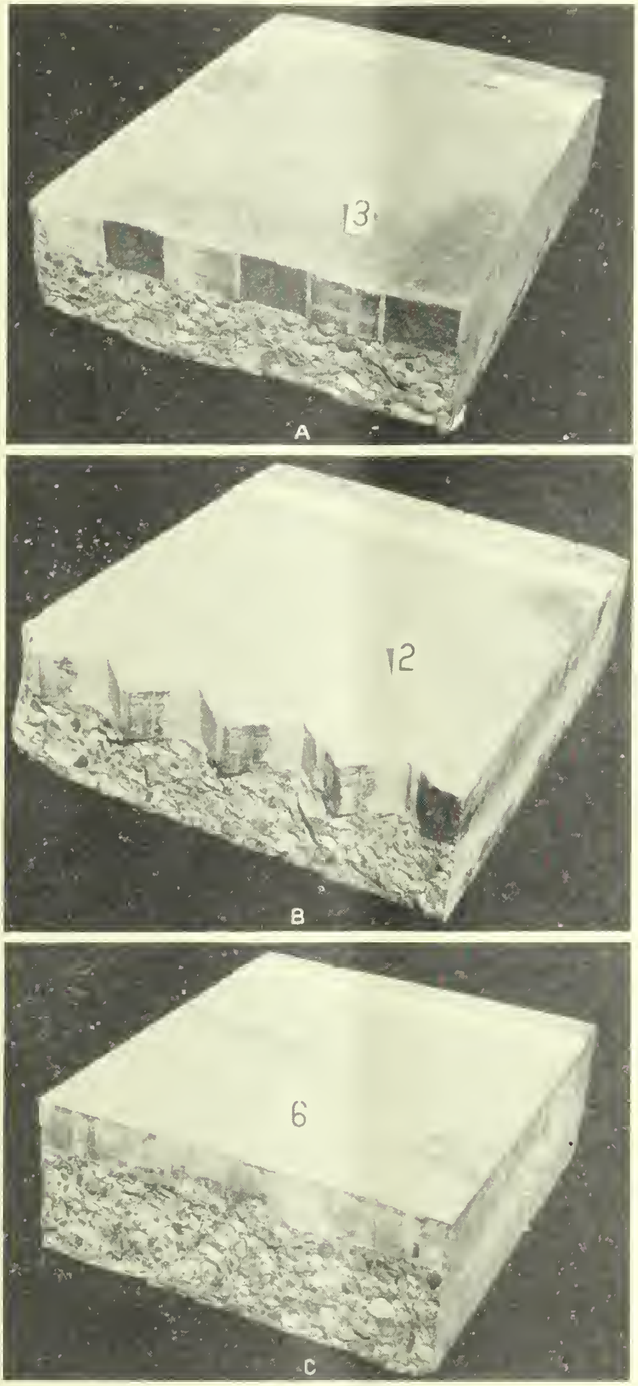


Figure 4.—Monolithic Brick Slabs Tested with Brick Surface in Tension. Brick Courses Laid (A) Perpendicular, (B) at an Angle of 45°, and (C) Parallel to the Axis of the Loading Knife Blade. Note Failure Between Brick and Mortar in (C).

perpendicular to the axis of the loading knife edge exhibited strengths equal to about 90 percent of the strength of plain concrete slabs of equal thickness, while the strength of the slabs in which the brick were set at a 45° angle averaged 67 percent of the strength of plain concrete slabs. In these tests the concrete used in the monolithic brick slabs was of the same quality (6 sacks of cement per cubic yard) as that used in the plain concrete slabs.

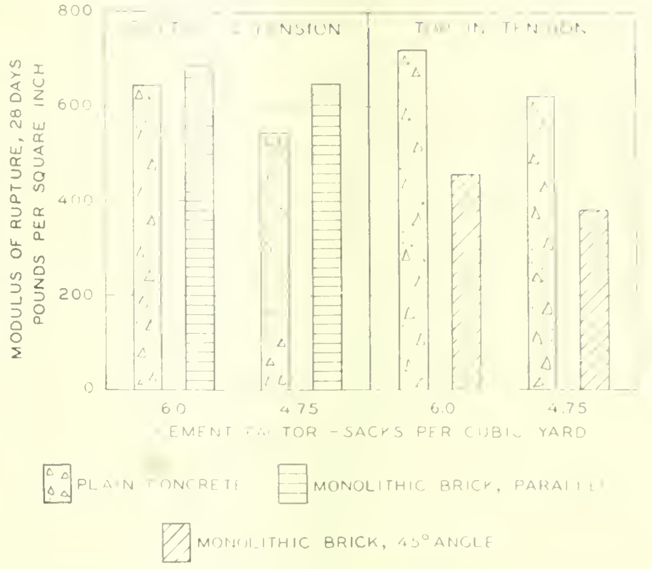


Figure 5.—Results of Flexure Tests on Monolithic Brick Pavement Slabs, Series B, at Age of 28 Days.

Strength results for series B are shown in table 7 and in figure 5. In this series all specimens were 8 inches thick. However, two different classes of concrete were used; one the pavement mix (6 sacks of cement per cubic yard), the other the base mix (4¾ sacks of cement per cubic yard). Strength results for slabs tested with the bottom, as cast, in tension are shown in the left panel of figure 5. It will be observed that, for both the pavement and base concretes, the monolithic brick slabs gave higher strengths than did the plain specimens, the average differential in strength being 13 percent. Referring to the results for series A as given in table 6, it will be observed that the differential in favor of the 8-inch monolithic brick slabs, tested with the bottom in tension was also 13 percent.

Table 7.—*Summary of results of flexure tests on monolithic brick pavement slabs, series B* [1]

Structure	Cement factor	Surface in tension	Direction of knife edge of applied load [2]	Flexural strength [3]	Strength ratio [4]
	Sacks per cu. yd.			*Lb. per sq. in.*	*Percent*
Concrete	6.0	Bottom		646	100
Monolithic brick	6.0	do	Parallel	686	106
Concrete	4.75	do		545	100
Monolithic brick	4.75	do	Parallel	650	119
Concrete	6.0	Top		721	100
Monolithic brick	6.0	do	45°	454	63
Concrete	4.75	do		622	100
Monolithic brick	4.75	do	45°	379	61

[1] Specimens 26 inches wide by 60 inches long by 8 inches thick (approximately); tested with center loading on a 54-inch span after 28 days of moist curing.

[2] With respect to the line of continuous brick courses.

[3] Flexural strength computed with neutral axis assumed at the center of the slab; each result the average of three tests.

[4] Plain concrete slabs were taken as standard.

The right panel in figure 5 gives results for slabs tested with the top surface, as cast, in tension, the courses of brick being placed at an angle of 45° to the axis of the loading knife edge. The effect of having the brick in tension was to lower the strength to 62 percent of that of plain concrete. This reduction in strength is also in close agreement with the results obtained in series A where the strength ratio was 60 percent for 8-inch monolithic brick slabs tested with the brick surface in tension and the brick courses at an angle of 45°.

The data plotted in figure 5 are of interest in giving a comparison of monolithic brick slabs using concrete of base mix proportions with plain concrete slabs of pavement proportions. As will be noted by comparing the first and fourth bars in the left panel of figure 5, the monolithic brick slabs using base mix proportions gave almost identical results with those obtained for the plain concrete slabs of pavement proportions. This, of course, was with the brick surface in compression. However, where the slabs were tested with the brick surface in tension, the monolithic brick slabs using base concrete were very much lower in strength.

Summarizing the data for series B, the results check very closely with those found in series A, where comparable tests were made. In addition, it was found that monolithic brick slabs made with base concrete were equal in strength to vibrated slabs of pavement concrete, when tested with the bottom in tension.

SUMMARY

The wheel load of a vehicle on a rigid pavement produces tensile stress in the bottom of the slab when the load is at the interior of the slab at some distance from an edge, or at the edge of the slab at some distance from a corner. Therefore, with respect to resistance to loads at the interior and edges of monolithic brick pavements, the most significant results of these tests were the strengths obtained with the slabs tested with the bottoms in tension.

On the other hand, a concentrated load applied at the corner of a rigid pavement produces tensile stress in the top of the slab and, if rupture occurs it will take place along a line that is roughly at an angle of 45° with the axis of the slab. Thus, if a monolithic brick pavement fails under a corner load, the slab will be ruptured along a line that is at an angle of approximately 45° with the direction of the courses of brick. The test slabs with the brick placed at an angle of 45° with the direction of the applied load were built to simulate this corner condition. When tested with the brick surface in tension they ruptured in the same direction, with respect to the brick courses, as the corner of a monolithic brick pavement would be expected to rupture. Therefore, of the results obtained with the brick slabs tested with the tops in tension, the most significant, with respect to their application in pavement design, are the strengths of the slabs with the brick placed at an angle of 45°.

With the above discussion in mind, the results of these tests may be summarized as follows, all comparisons being made on the basis of slabs of equal thickness:

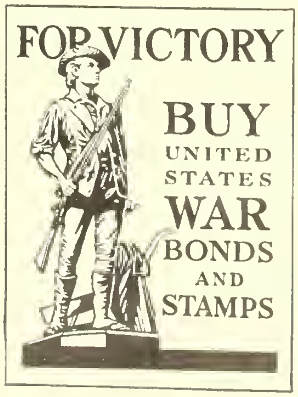
1. Monolithic brick slabs, when tested with the concrete in tension, developed flexural strengths ranging from 106 to 119 percent of the strengths of comparable slabs of plain concrete.
2. Monolithic brick slabs of base concrete and plain concrete slabs of pavement concrete, when tested with the concrete in tension, developed approximately the same strengths.
3. In monolithic brick slabs tested with the concrete in tension, the bond between the brick surface and the concrete base apparently was not affected by 100 applications of a load equal to 50 percent of the ultimate load.
4. Monolithic brick slabs with the brick at an angle of 45° with the axis of the slab and tested with the brick surface in tension developed flexural strengths ranging from 60 to 76 percent of the strengths of comparable slabs of plain concrete.
5. Monolithic brick slabs of base concrete, with the brick at an angle of 45° with the axis of the slab and tested with the brick surface in tension, developed about 53 percent of the strength of plain concrete slabs of pavement concrete.

HIGHWAY RESEARCH BOARD MEETS IN ST. LOUIS, DECEMBER 2–4, 1942

The Twenty-second Annual Meeting of the Highway Research Board will be held December 2–4, 1942, at the Hotel Statler in St. Louis, Mo.

The customary sessions of the board for the discussion of topics relating to highway finance, economics, design, materials, construction, maintenance, traffic, and soils investigation will be held.

It is expected that time and travel facilities will be greatly conserved by holding this meeting contiguous to that of the American Association of State Highway Officials which will be held in St. Louis the following week.



MOTOR-FUEL CONSUMPTION-1941

COMPILED FOR CALENDAR YEAR FROM REPORTS OF STATE AUTHORITIES 1/

TABLE G-2, 1941
ISSUED JUNE 1942

STATE	TAX RATE PER GALLON ON DECEMBER 31	GROSS AMOUNT REPORTED 2/	AMOUNT EXEMPTED FROM PAYMENT OF TAX 3/	GROSS AMOUNT ASSESSED FOR TAXATION	AMOUNT SUBJECT TO REFUND OF ENTIRE TAX	NET AMOUNT TAXED				AMOUNT TAXED AT PREVAILING RATE DURING 1940	INCREASE DURING 1941		STATE
						TOTAL	AT PREVAILING RATE	AT OTHER RATES			AMOUNT	PERCENTAGE	
								RATE PER GALLON	AMOUNT				
	CENTS	1,000 GALLONS	1,000 GALLONS	1,000 GALLONS	1,000 GALLONS	1,000 GALLONS	1,000 GALLONS	CENTS	1,000 GALLONS	1,000 GALLONS	1,000 GALLONS		
ALABAMA	6	310,603	-	310,603	-	310,603	310,603	-	-	259,915	50,688	19.5	ALABAMA
ARIZONA	5	130,954	8,060	122,894	12,883	110,011	110,011	-	-	95,707	14,304	14.9	ARIZONA
ARKANSAS	6.5	223,634	8,872	214,762	-	214,762	190,095	(4/)	24,667	162,328	27,767	17.1	ARKANSAS
CALIFORNIA	3	2,272,732	119,055	2,153,677	175,130	1,978,547	1,978,547	-	-	1,758,326	220,221	12.5	CALIFORNIA
COLORADO	4	270,287	14,279	256,008	34,489	221,519	221,519	-	-	206,742	14,777	7.1	COLORADO
CONNECTICUT	3	422,586	14,302	408,284	9,735	398,549	398,549	-	-	364,594	33,955	9.3	CONNECTICUT
DELAWARE	4	69,122	1,837	67,285	4,197	63,088	63,088	-	-	58,397	4,691	8.0	DELAWARE
FLORIDA	7	475,905	49,446	426,459	-	426,459	426,459	-	-	388,717	37,742	9.7	FLORIDA
GEORGIA	6	449,335	22,277	427,058	-	427,058	427,058	-	-	386,847	40,211	10.4	GEORGIA
IDAHO	5.1	117,232	4,124	113,108	-	113,108	102,255	(5/)	10,853	94,565	7,690	8.1	IDAHO
ILLINOIS	3	1,670,862	-	1,670,862	135,385	1,535,477	1,535,477	-	-	1,410,967	124,510	8.8	ILLINOIS
INDIANA	4	799,091	7,484	791,607	70,074	721,533	721,533	-	-	641,164	80,369	12.5	INDIANA
IOWA	3	612,221	-	612,221	93,072	519,149	519,149	-	-	485,995	33,154	6.8	IOWA
KANSAS	3	541,494	162,862	378,632	-	378,632	378,632	-	-	346,720	31,912	9.2	KANSAS
KENTUCKY	5	352,743	15,527	337,216	-	337,216	337,216	-	-	295,205	42,011	14.2	KENTUCKY
LOUISIANA	7	343,599	29,263	314,336	6	314,330	304,900	2	6/ 9,430	264,057	40,843	15.5	LOUISIANA
MAINE	4	173,874	1,461	172,413	-	172,413	163,942	1	7/ 8,471	149,130	14,812	9.9	MAINE
MARYLAND	4	365,530	10,006	355,524	22,626	332,898	329,903	3	8/ 2,995	286,636	43,267	15.1	MARYLAND
MASSACHUSETTS	3	801,957	9,609	792,348	34,277	758,071	758,071	-	-	713,358	44,713	6.3	MASSACHUSETTS
MICHIGAN	3	1,392,274	128,012	1,264,262	64,764	1,199,498	1,198,472	1.5	9/ 1,026	1,082,006	116,466	10.8	MICHIGAN
MINNESOTA	10/ 4	625,173	33,640	591,533	71,526	520,007	11/ 520,007	-	-	497,556	22,451	4.5	MINNESOTA
MISSISSIPPI	6	254,877	14,897	239,980	-	239,980	232,458	1	12/ 7,522	197,570	34,888	17.7	MISSISSIPPI
MISSOURI	2	776,432	-	776,432	38,710	737,722	737,722	-	-	658,321	79,401	12.1	MISSOURI
MONTANA	5	148,262	6,717	141,545	28,296	113,249	113,249	-	-	104,451	8,798	8.4	MONTANA
NEBRASKA	5	259,765	11,255	248,510	21	248,489	248,339	1	13/ 150	233,438	14,901	6.4	NEBRASKA
NEVADA	4	49,018	2,909	46,109	3,108	43,001	40,807	5	14/ 2,194	35,717	5,090	14.3	NEVADA
NEW HAMPSHIRE	4	102,395	896	101,499	5,055	96,444	96,444	-	-	90,822	5,622	6.2	NEW HAMPSHIRE
NEW JERSEY	3	1,006,334	42,230	964,104	85,920	878,184	878,184	-	-	822,426	55,758	6.8	NEW JERSEY
NEW MEXICO	5	122,275	6,986	115,289	12,221	103,068	103,068	-	-	94,196	8,872	9.4	NEW MEXICO
NEW YORK	4	2,058,071	90,529	1,967,542	69,061	1,898,481	1,898,481	-	-	1,836,977	61,504	3.3	NEW YORK
NORTH CAROLINA	6	526,341	-	526,341	-	526,341	511,064	1	12/ 15,277	440,548	70,516	16.0	NORTH CAROLINA
NORTH DAKOTA	4	160,815	70,770	90,045	-	90,045	90,045	-	-	85.780	4,265	5.0	NORTH DAKOTA
OHIO 15/	4	1,638,913	90,737	1,548,176	18,675	1,529,501	1,446,626	1	7/ 82,875	1,320,885	125,741	9.5	OHIO
OKLAHOMA	16/ 5.5	473,597	78,555	395,042	-	395,042	17/ 395,042	-	-	378,275	16,767	4.4	OKLAHOMA
OREGON	5	306,484	9,348	297,136	31,167	265,969	264,132	1	18/ 1,837	228,396	35,736	15.6	OREGON
PENNSYLVANIA	4	1,704,947	11,201	1,693,746	-	1,693,746	1,693,746	-	-	1,575,078	118,668	7.5	PENNSYLVANIA
RHODE ISLAND	3	147,280	3,605	143,675	1,445	142,230	142,230	-	-	131,558	10,672	8.1	RHODE ISLAND
SOUTH CAROLINA	6	275,514	-	275,514	9,416	266,098	266,098	-	-	229,398	36,700	16.0	SOUTH CAROLINA
SOUTH DAKOTA	4	156,036	2,681	153,355	33,804	119,551	119,551	-	-	113,659	5,892	5.2	SOUTH DAKOTA
TENNESSEE	7	386,300	26,316	359,984	4,955	355,029	355,029	-	-	304,042	50,987	16.8	TENNESSEE
TEXAS	4	1,664,602	101,499	1,563,103	221,435	1,341,668	1,341,668	-	-	1,195,694	145,974	12.2	TEXAS
UTAH	4	118,062	4,356	113,706	-	113,706	113,706	-	-	102,425	11,281	11.0	UTAH
VERMONT	4	75,189	944	74,245	-	74,245	74,245	-	-	69,926	4,319	6.2	VERMONT
VIRGINIA	5	503,281	-	503,281	31,484	471,797	471,019	3	19/ 778	392,386	78,633	20.0	VIRGINIA
WASHINGTON	5	450,676	28,855	421,821	28,753	393,068	393,068	-	-	347,872	45,196	13.0	WASHINGTON
WEST VIRGINIA	5	242,167	-	242,167	8,227	233,940	233,940	-	-	214,983	18,957	8.8	WEST VIRGINIA
WISCONSIN	4	636,760	18,844	617,916	42,334	575,582	575,582	-	-	529,733	45,849	8.7	WISCONSIN
WYOMING	4	79,840	2,531	77,309	-	77,309	77,309	-	-	68,999	8.310	12.0	WYOMING
DISTRICT OF COLUMBIA	20/ 2	191,920	9,758	182,162	3,578	178,584	178,584	-	-	160,954	17,630	11.0	DISTRICT OF COLUMBIA
TOTAL	21/ 3.99	26,937,361	1,276,535	25,660,826	1,405,829	24,254,997	24,086,922	-	168,075	21,913,441	2,173,481	9.9	TOTAL

1/ AN ANALYSIS OF MOTOR-FUEL USAGE WILL BE GIVEN IN TABLE G-21, TO BE PUBLISHED LATER.

2/ EXPORT SALES AND OTHER AMOUNTS NOT REPRESENTING CONSUMPTION IN STATE HAVE BEEN ELIMINATED AS FAR AS POSSIBLE. IN CASES WHERE STATES FAILED TO REPORT AMOUNTS EXEMPTED FROM TAXATION, THE GROSS AMOUNT TAXED IS SHOWN IN THIS COLUMN.

3/ INCLUDES ALLOWANCES FOR EVAPORATION AND OTHER LOSSES, FEDERAL USE, OTHER PUBLIC USE, AND NONHIGHWAY USE, WHERE INITIAL EXEMPTIONS RATHER THAN REFUNDS ARE MADE.

4/ WITHIN 300 FEET OF BORDER, TAX IS REDUCED TO THAT OF ADJACENT STATE. GALLONS TAXED AT 2 CENTS, 6,731,000; AT 4 CENTS, 10,019,000; AT 5.5 CENTS, 7,917,000.

5/ AVIATION FUEL TAXED AT 2.5 CENTS, 607,000 GALLONS; MOTOR FUEL TAXED AT 0.1 CENT (5 CENTS REFUNDED ON NONHIGHWAY USE), 10,246,000 GALLONS.

6/ REPRESENTS EVAPORATION OR LOSS ALLOWANCE UNDER 5-CENT TAX NOT ALLOWED UNDER ADDITIONAL 2-CENT TAX, WHICH IS ADMINISTERED UNDER A SEPARATE LAW.

7/ THREE CENTS PER GALLON REFUNDED ON NONHIGHWAY USES.

8/ ONE CENT PER GALLON REFUNDED ON MOTOR FUEL USED IN VEHICLES LICENSED TO OPERATE EXCLUSIVELY IN CITIES.

9/ ONE AND ONE-HALF CENTS PER GALLON REFUNDED ON MOTOR FUEL USED IN INTERSTATE AVIATION.

10/ RATE CHANGED FROM 3 CENTS TO 4 CENTS MAY 1.

11/ TAXED AT 3 CENTS, 142,044,000 GALLONS; AT 4 CENTS, 377,963,000 GALLONS.

12/ FIVE CENTS PER GALLON REFUNDED ON NONHIGHWAY USES.

13/ FOUR CENTS PER GALLON REFUNDED ON MOTOR FUEL USED IN FLYING INSTRUCTION.

14/ DIESEL FUEL AND BUTANE.

15/ AMOUNTS GIVEN DO NOT INCLUDE 63,317,000 GALLONS OF LIQUID FUEL (KEROSENE, FUEL OIL, ETC.) TAXED AT 1 CENT PER GALLON BUT NOT SUBJECT TO THE 3-CENT TAX ON MOTOR-VEHICLE FUEL.

16/ RATE CHANGED FROM 4 CENTS TO 5.5 CENTS JUNE 1.

17/ TAXED AT 4 CENTS, 163,980,000 GALLONS; AT 5.5 CENTS, 231,062,000 GALLONS.

18/ FOUR CENTS PER GALLON REFUNDED ON MOTOR FUEL USED IN AVIATION.

19/ TWO CENTS PER GALLON REFUNDED ON MOTOR FUEL USED IN INTRASTATE AVIATION.

20/ RATE CHANGED FROM 2 CENTS TO 3 CENTS JANUARY 1, 1942.

21/ WEIGHTED AVERAGE RATE.

STATE MOTOR-FUEL TAX RECEIPTS - 1941

COMPILED FOR CALENDAR YEAR FROM REPORTS OF STATE AUTHORITIES

TABLE C-1, 1941
ISSUED JUNE 1942

STATE	TAX RATE PER GALLON ON DECEMBER 31	RECEIPTS FROM TAXATION OF MOTOR FUEL					OTHER RECEIPTS IN CONNECTION WITH MOTOR-FUEL TAX 2/					NET TOTAL RECEIPTS	LESS TAX ON AVIATION GASOLINE	ADJUSTED NET TOTAL RECEIPTS	STATE
		GROSS TAX COLLECTIONS	DEDUCTIONS BY DISTRIBUTORS FOR EXPENSES 1/	GROSS RECEIPTS BY STATE	REFUNDS PAID	NET RECEIPTS BY STATE	DISTRIBUTORS AND DEALERS LICENSES	INSPECTION FEES 3/	FINES AND PENALTIES	MISCELLANEOUS RECEIPTS	TOTAL				
	CENTS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	
ALABAMA	6	18,323	-	18,323	-	18,323	-	78	-	*	78	18,401	-	18,401	ALABAMA
ARIZONA	5	6,148	-	6,148	787	5,361	*	-	1	-	1	5,362	-	5,362	ARIZONA
ARKANSAS	6.5	13,100	-	13,100	-	13,100	-	119	-	-	119	13,219	-	13,219	ARKANSAS
CALIFORNIA	3	63,330	-	63,330	5,254	58,076	17	-	-	1	18	58,094	-	58,094	CALIFORNIA
COLORADO	4	10,195	-	10,195	1,362	8,833	-	-	-	-	-	8,833	-	8,833	COLORADO
CONNECTICUT	3	12,180	122	12,058	300	11,758	45	-	1	-	46	11,804	-	11,804	CONNECTICUT
DELAWARE	4	2,665	-	2,665	164	2,501	3	-	-	-	3	2,504	-	2,504	DELAWARE
FLORIDA	7	29,832	-	29,832	-	29,832	41	524	-	-	565	30,397	-	30,397	FLORIDA
GEORGIA	6	26,311	262	26,049	-	26,049	37	-	-	-	37	26,086	-	26,086	GEORGIA
IDAHO	5.1	5,722	-	5,722	491	5,231	1	-	-	*	1	5,232	14	5,218	IDAHO
ILLINOIS	3	49,656	993	48,663	3,901	44,762	-	498	1	-	499	45,261	-	45,261	ILLINOIS
INDIANA	4	31,270	-	31,270	2,752	28,518	*	700	24	2	726	29,244	-	29,244	INDIANA
IOWA	3	18,199	-	18,199	2,861	15,338	62	-	-	-	62	15,400	-	15,400	IOWA
KANSAS	3	11,234	-	11,234	-	11,234	15	118	-	22	155	11,389	-	11,389	KANSAS
KENTUCKY	5	16,338	163	16,175	-	16,175	-	-	5	-	5	16,180	-	16,180	KENTUCKY
LOUISIANA	7	21,425	-	21,425	-	21,425	-	107	30	-	137	21,562	-	21,562	LOUISIANA
MAINE	4	6,918	-	6,918	261	6,657	-	-	*	*	*	6,657	3	6,654	MAINE
MARYLAND	4	14,000	-	14,000	930	13,070	-	-	-	-	-	13,070	-	13,070	MARYLAND
MASSACHUSETTS	3	23,576	-	23,576	1,088	22,488	48	-	*	-	48	22,536	-	22,536	MASSACHUSETTS
MICHIGAN	3	37,586	-	37,586	2,001	35,585	5	-	2	-	7	35,592	88	35,504	MICHIGAN
MINNESOTA	4/ 4	21,570	-	21,570	2,700	18,870	*	182	3	*	185	19,055	-	19,055	MINNESOTA
MISSISSIPPI 5/	6	14,126	-	14,126	373	13,753	-	-	-	-	-	13,753	-	13,753	MISSISSIPPI 5/
MISSOURI	2	15,252	-	15,252	975	14,277	-	165	9	-	174	14,451	-	14,451	MISSOURI
MONTANA	5	6,878	-	6,878	1,411	5,467	-	4	-	-	4	5,471	-	5,471	MONTANA
NEBRASKA	5	12,711	94	12,617	349	12,268	11	128	-	34	173	12,441	84	12,357	NEBRASKA
NEVADA	4	1,842	34	1,808	125	1,683	*	24	1	-	25	1,708	-	1,708	NEVADA
NEW HAMPSHIRE	4	3,913	-	3,913	198	3,715	-	-	1	-	1	3,716	-	3,716	NEW HAMPSHIRE
NEW JERSEY	3	28,814	-	28,814	2,593	26,221	69	-	2	14	85	26,306	-	26,306	NEW JERSEY
NEW MEXICO	5	5,695	-	5,695	619	5,076	26	-	-	-	26	5,102	-	5,102	NEW MEXICO
NEW YORK	4	78,041	780	77,261	2,803	74,458	62	-	-	*	62	74,520	-	74,520	NEW YORK
NORTH CAROLINA	6	31,417	-	31,417	768	30,649	-	1,271	-	11	1,282	31,931	-	31,931	NORTH CAROLINA
NORTH DAKOTA	4	3,569	54	3,515	-	3,515	1	83	-	26	110	3,625	-	3,625	NORTH DAKOTA
OHIO	4	6/ 61,274	-	61,274	3,161	58,113	-	-	-	-	-	58,113	-	58,113	OHIO
OKLAHOMA	7/ 5.5	19,141	478	18,663	2	18,661	-	315	-	-	315	18,976	-	18,976	OKLAHOMA
OREGON	5	14,702	-	14,702	1,673	13,029	-	-	*	-	*	13,029	18	13,011	OREGON
PENNSYLVANIA	4	65,627	760	64,867	-	64,867	-	-	5	-	5	64,872	-	64,872	PENNSYLVANIA
RHODE ISLAND	3	4,627	-	4,627	355	4,272	3	-	-	-	3	4,275	-	4,275	RHODE ISLAND
SOUTH CAROLINA	6	16,356	-	16,356	539	15,817	-	330	-	-	330	16,147	87	16,060	SOUTH CAROLINA
SOUTH DAKOTA	4	6,112	215	5,897	1,338	4,559	-	79	-	-	79	4,638	21	4,617	SOUTH DAKOTA
TENNESSEE	7	24,505	-	24,505	315	24,190	-	1,386	-	69	1,455	25,645	246	25,399	TENNESSEE
TEXAS	4	62,972	633	62,339	9,592	52,747	-	-	-	21	21	52,768	-	52,768	TEXAS
UTAH	4	4,500	68	4,432	-	4,432	1	-	1	*	2	4,434	64	4,370	UTAH
VERMONT	4	2,930	-	2,930	-	2,930	-	-	*	-	*	2,930	-	2,930	VERMONT
VIRGINIA	5	24,501	-	24,501	1,585	22,916	1	-	2	*	3	22,919	23	22,896	VIRGINIA
WASHINGTON	5	20,844	104	20,740	1,419	19,321	3	-	-	10	13	19,334	-	19,334	WASHINGTON
WEST VIRGINIA	5	11,956	-	11,956	403	11,553	6	-	-	-	6	11,559	-	11,559	WEST VIRGINIA
WISCONSIN	4	24,432	-	24,432	1,695	22,737	-	189	-	-	189	22,926	-	22,926	WISCONSIN
WYOMING	4	3,065	-	3,065	-	3,065	2	-	-	-	2	3,067	53	3,014	WYOMING
DISTRICT OF COLUMBIA	8/ 2	3,550	-	3,550	71	3,479	*	-	-	-	*	3,479	-	3,479	DISTRICT OF COLUMBIA
TOTAL	9/ 3.99	1,012,930	4,760	1,008,170	57,214	950,956	459	6,300	88	210	7,057	958,013	701	957,312	TOTAL

1/ THE STATES FOR WHICH AMOUNTS ARE SHOWN MAKE ALLOWANCES TO DISTRIBUTORS FOR EXPENSE OF COLLECTING THE TAX. IN KENTUCKY, SOUTH DAKOTA, UTAH, AND WASHINGTON ALLOWANCES OF 2 1/4, 4, 3, AND 1 PERCENT, RESPECTIVELY, OF THE TAX OTHERWISE DUE ARE MADE IN CONSIDERATION OF BOTH EXPENSE OF COLLECTION AND GALLONAGE LOSSES IN HANDLING. IN THESE STATES THE ALLOWANCES FOR EXPENSES ONLY HAVE BEEN ESTIMATED AS 1, 3½, 1½, AND ½ PERCENT, RESPECTIVELY.

2/ STARS INDICATE AMOUNTS LESS THAN $500.

3/ FEES FOR INSPECTION OF MOTOR-VEHICLE FUEL. WHEREVER POSSIBLE, FEES FOR INSPECTION OF KEROSENE AND OTHER NON-MOTOR-VEHICLE FUELS HAVE BEEN ELIMINATED.

4/ RATE CHANGED FROM 3 CENTS TO 4 CENTS MAY 1.

5/ SPECIAL COUNTY TAXES OF 3 CENTS PER GALLON IN HANCOCK COUNTY AND 2 CENTS PER GALLON IN HARRISON AND JACKSON COUNTIES, AMOUNTING TO $306,000 IN 1941, ARE IMPOSED FOR SEAWALL PROTECTION AND ARE NOT INCLUDED IN THIS TABLE.

6/ OHIO IMPOSES A 3-CENT TAX ON MOTOR-VEHICLE FUEL AND A 1-CENT TAX ON ALL LIQUID FUELS. THE RECEIPTS FROM THE 1-CENT TAX APPLICABLE TO NON-MOTOR-VEHICLE FUELS (KEROSENE, FUEL OIL, ETC.) WERE $627,000. THESE RECEIPTS HAVE BEEN ELIMINATED FROM THE TOTAL GIVEN, WHICH REPRESENTS A 4-CENT TAX ON MOTOR-VEHICLE FUEL.

7/ RATE CHANGED FROM 4 CENTS TO 5.5 CENTS JUNE 1.

8/ THIS RATE NO LONGER IN EFFECT; RATE CHANGED FROM 2 CENTS TO 3 CENTS JANUARY 1, 1942.

9/ WEIGHTED AVERAGE RATE.

TABLE MV-1, 1941
ISSUED JUNE 1942

STATE MOTOR-VEHICLE REGISTRATIONS-1941

COMPILED FOR CALENDAR YEAR FROM REPORTS OF STATE AUTHORITIES 1/

STATE	MOTOR VEHICLES									TRAILERS AND SEMITRAILERS				MOTORCYCLES				DEALERS REGISTRATIONS 7/		1940 TOTAL REGISTERED MOTOR VEHICLES, PRIVATE AND COMMERCIAL ONLY	YEAR'S CHANGE IN MOTOR-VEHICLE REGISTRATIONS		STATE
	TOTAL	PRIVATE AND COMMERCIAL 2/					PUBLICLY OWNED			TOTAL	PRIVATE AND COMMERCIAL 6/	PUBLICLY OWNED		TOTAL	PRIVATE AND COMMERCIAL 2/	PUBLICLY OWNED		REGULAR	EXTRA PLATES		INCREASE	PERCENTAGE INCREASE	
		TOTAL	PASSENGER VEHICLES			TRUCKS AND TRACTOR TRUCKS	TOTAL	FEDERAL 4/	STATE, COUNTY, AND MUNICIPAL 5/			FEDERAL 4/	STATE, COUNTY, AND MUNICIPAL 5/			FEDERAL 4/	STATE, COUNTY, AND MUNICIPAL 5/						
			TOTAL	AUTOMOBILES (INCLUDING TAXICABS)	BUSSES 3/																		
ALABAMA	414,451	409,706	335,000	330,727	4,273	74,706	4,745	1,780	2,965	6,526	6,478	48	-	1,400	1,400	-	-	3,185	-	339,853	69,853	20.6	ALABAMA
ARIZONA	149,440	144,401	117,712	117,377	335	26,689	5,039	2,689	2,350	5,573	5,321	97	155	574	543	1	30	334	1,574	138,403	5,998	4.3	ARIZONA
ARKANSAS	294,531	290,249	213,058	212,522	536	77,191	4,282	1,607	2,675	14,836	14,781	10	45	742	706	1	35	866	2,989	257,177	33,072	12.9	ARKANSAS
CALIFORNIA	2,996,775	2,962,222	2,618,369	2,614,106	4,263	343,853	34,553	5,767	28,786	186,258	184,104	114	2,040	19,400	18,063	4	1,333	4,918	-	2,773,659	188,563	6.8	CALIFORNIA
COLORADO	370,180	367,768	307,402	306,182	1,220	60,366	2,412	2,412	-	1,918	1,887	31	-	1,310	1,308	2	-	4,548	-	352,110	15,658	4.4	COLORADO
CONNECTICUT	555,838	551,101	471,845	470,566	1,279	79,256	4,737	488	4,249	8,095	7,857	2	236	2,607	2,312	-	295	3,045	9,893	493,667	57,434	11.6	CONNECTICUT
DELAWARE	77,730	76,842	65,014	64,662	352	11,828	888	127	761	3,339	3,274	2	63	396	342	-	54	983	-	71,763	5,079	7.1	DELAWARE
FLORIDA	553,991	545,706	460,468	458,687	1,781	85,238	8,285	1,773	6,512	24,220	23,802	25	393	2,180	2,004	1	175	2,409	-	495,210	50,496	10.2	FLORIDA
GEORGIA	566,212	558,984	463,921	460,787	3,134	95,063	7,228	2,091	5,137	16,609	16,528	22	59	1,951	1,777	6	168	571	3,127	502,603	56,381	11.2	GEORGIA
IDAHO	174,354	171,031	134,612	134,473	139	36,419	3,323	1,437	1,886	22,813	22,630	71	112	608	594	1	13	424	-	163,340	7,691	4.7	IDAHO
ILLINOIS	2,062,884	2,048,805	1,826,583	1,823,884	2,699	222,222	14,079	3,039	11,040	32,263	31,159	27	1,077	7,673	7,148	1	524	4,862	-	1,925,814	122,991	6.4	ILLINOIS
INDIANA	1,087,648	1,078,954	934,866	928,399	6,467	144,088	8,694	1,114	7,580	83,042	83,031	11	-	6,348	6,126	-	222	2,789	-	994,006	84,948	8.5	INDIANA
IOWA	834,103	825,112	715,108	714,608	500	110,004	8,991	1,184	7,807	102,495	101,944	6	545	2,843	2,843	-	-	2,047	5,177	793,969	31,143	3.9	IOWA
KANSAS	618,480	617,793	504,581	503,921	660	113,212	687	687	-	6,065	5,953	112	-	1,418	1,418	-	-	2,281	3,048	582,668	35,125	6.0	KANSAS
KENTUCKY	503,781	497,427	415,764	414,845	919	81,663	6,354	1,302	5,052	15	(8/)	15	-	1,315	1,315	-	-	919	-	463,727	33,700	7.3	KENTUCKY
LOUISIANA	437,214	430,707	348,337	344,947	3,390	82,370	6,507	1,416	5,091	16,305	16,273	31	1	1,796	1,720	1	75	242	-	365,429	65,278	17.9	LOUISIANA
MAINE	225,319	222,367	176,619	176,321	298	9/ 45,748	2,952	456	2,496	12,572	9/ 12,344	-	228	821	786	4	31	548	3,797	205,896	16,471	8.0	MAINE
MARYLAND	497,998	494,141	429,937	428,764	1,173	64,204	3,857	1,541	2,316	7,241	6,858	313	70	2,196	2,142	2	52	11,535	-	444,532	49,609	11.2	MARYLAND
MASSACHUSETTS	963,122	961,639	850,989	845,874	5,115	110,650	1,483	1,483	-	18,711	18,695	16	-	1,194	1,194	-	-	2,716	20,217	903,843	57,796	6.4	MASSACHUSETTS
MICHIGAN	1,707,458	1,705,610	1,544,245	10/ 1,543,255	990	10/ 161,365	1,848	1,848	-	169,433	169,403	30	-	5,071	5,071	-	-	2,091	2,091	1,552,561	153,049	9.9	MICHIGAN
MINNESOTA	909,903	903,042	773,332	772,708	624	129,710	6,861	1,703	5,158	118,204	117,848	36	320	2,594	2,467	-	127	3,703	-	871,351	31,691	3.6	MINNESOTA
MISSISSIPPI	299,047	293,725	222,665	220,807	1,858	71,060	5,322	1,408	3,914	8,137	8,111	26	-	482	452	-	30	2,709	-	259,013	34,712	13.4	MISSISSIPPI
MISSOURI	992,505	984,626	823,158	820,080	3,078	161,468	7,879	1,887	5,992	46,100	46,052	14	34	2,476	2,450	1	25	2,831	3,942	921,800	62,826	6.8	MISSOURI
MONTANA	204,122	198,732	147,606	147,256	350	51,126	5,390	2,469	2,921	6,630	6,544	86	-	523	523	-	-	1,053	-	191,032	7,700	4.0	MONTANA
NEBRASKA	427,509	423,058	354,598	354,276	322	68,460	4,451	1,013	3,438	48,262	47,743	26	493	1,321	1,259	-	62	2,012	2,186	412,116	10,942	2.7	NEBRASKA
NEVADA	49,868	48,160	38,636	38,480	156	9,524	1,708	902	806	1,935	1,797	49	89	178	169	-	9	86	318	44,108	4,052	9.2	NEVADA
NEW HAMPSHIRE	142,685	142,404	110,286	109,971	315	32,118	281	281	-	6,962	6,958	4	-	746	743	3	-	556	-	135,384	7,020	5.2	NEW HAMPSHIRE
NEW JERSEY	1,178,438	1,165,916	1,024,587	1,019,155	5,432	141,329	12,522	1,486	11,036	9,211	9,197	14	-	6,066	5,581	-	485	3,163	18,966	1,086,966	78,950	7.3	NEW JERSEY
NEW MEXICO	132,913	129,211	98,405	97,127	1,278	30,806	3,702	2,292	1,410	3,490	3,414	76	-	440	397	4	39	161	-	125,056	4,155	3.3	NEW MEXICO
NEW YORK	2,892,201	2,859,929	2,519,066	2,510,169	8,897	340,863	32,272	3,885	28,387	57,492	56,296	115	1,081	12,321	11,111	4	1,206	5,350	-	2,743,014	116,915	4.3	NEW YORK
NORTH CAROLINA	678,522	661,779	563,357	562,017	1,340	98,422	16,743	2,297	14,446	47,803	47,776	27	-	1,910	1,910	-	-	11,243	-	591,946	69,833	11.8	NORTH CAROLINA
NORTH DAKOTA	194,594	192,895	152,107	151,977	130	40,788	1,699	796	903	1,903	1,887	16	-	230	230	-	-	655	448	182,287	10,608	5.8	NORTH DAKOTA
OHIO	2,018,802	1,996,000	1,801,800	1,800,000	1,800	194,200	22,802	2,052	20,750	126,279	125,000	29	1,250	9,932	9,500	2	430	4,443	20,366	1,918,929	77,071	4.0	OHIO
OKLAHOMA	601,368	591,822	479,363	476,566	2,797	112,459	9,546	1,836	7,710	8,909	8,876	33	-	1,211	1,211	-	-	4,146	-	574,719	17,103	3.0	OKLAHOMA
OREGON	436,111	429,440	353,902	353,213	689	11/ 75,538	6,671	2,022	4,649	56	(11/)	56	-	1,887	1,881	6	-	711	912	393,578	35,862	9.1	OREGON
PENNSYLVANIA	2,309,068	2,285,083	2,016,420	2,010,117	6,303	268,663	23,985	2,716	21,269	37,489	36,525	306	658	14,093	13,253	6	834	36,068	-	2,145,849	139,234	6.5	PENNSYLVANIA
RHODE ISLAND	200,743	198,866	178,281	177,780	501	20,585	1,877	292	1,585	1,430	1,420	1	9	976	883	-	93	394	-	187,509	11,357	6.1	RHODE ISLAND
SOUTH CAROLINA	396,023	389,498	336,401	334,884	1,517	53,097	6,525	1,539	4,986	6,323	6,301	22	-	1,300	1,077	-	223	1,401	2,788	336,772	52,726	15.7	SOUTH CAROLINA
SOUTH DAKOTA	205,665	202,875	167,796	167,655	141	35,079	2,790	1,246	1,544	25,935	25,782	15	138	457	444	-	13	785	-	195,667	7,208	3.7	SOUTH DAKOTA
TENNESSEE	518,948	508,983	427,961	424,911	3,050	11/ 81,022	9,965	2,955	7,010	80	(11/)	80	-	1,856	1,854	2	-	406	-	447,983	61,000	13.6	TENNESSEE
TEXAS	1,833,362	1,811,101	1,441,998	1,440,996	1,002	369,103	22,261	3,646	18,615	62,259	61,397	74	788	5,969	4,741	3	325	3,872	-	1,694,113	116,988	6.9	TEXAS
UTAH	153,522	150,493	126,264	125,633	631	24,229	3,029	1,350	1,679	732	631	26	75	565	528	2	35	436	1,201	139,921	10,572	7.6	UTAH
VERMONT	97,735	97,486	87,159	12/ 87,048	111	12/ 10,327	249	249	-	2,554	2,551	3	-	444	444	-	-	627	-	93,645	3,841	4.1	VERMONT
VIRGINIA	579,119	570,567	484,588	482,838	1,750	85,979	8,552	2,106	6,446	14,206	14,024	19	163	2,946	2,773	1	172	7,057	-	498,838	71,729	14.4	VIRGINIA
WASHINGTON	627,811	617,030	522,258	520,599	1,659	94,772	10,781	2,701	8,080	27,680	27,259	88	333	2,608	2,473	1	134	-	-	562,495	54,535	9.7	WASHINGTON
WEST VIRGINIA	341,702	335,840	280,539	279,700	839	55,301	5,862	990	4,872	4,201	4,125	20	56	1,400	1,358	-	42	14,278	-	302,558	33,282	11.0	WEST VIRGINIA
WISCONSIN	980,540	968,512	808,726	807,810	916	159,786	12,028	1,837	10,191	9,224	8,921	40	263	3,885	3,590	2	293	2,269	-	900,915	67,597	7.5	WISCONSIN
WYOMING	93,646	91,459	71,157	71,017	140	20,302	2,187	1,149	1,038	11,578	11,442	31	105	296	296	-	-	390	-	85,657	5,802	6.8	WYOMING
DIST. OF COL.	177,015	174,070	160,267	158,616	1,651	13,803	2,945	1,382	13/ 1,563	956	837	10	109	796	654	67	75	168	3,659	161,914	12,156	7.5	DIST. OF COL.
TOTAL	34,764,996	34,383,167	29,507,113	29,418,313	88,800	4,876,054	381,829	84,728	297,101	1,434,349	1,421,036	2,325	10,988	140,851	133,064	128	7,659	162,286	106,699	32,025,365	2,357,802	7.4	TOTAL

1/ REGISTRATION PERIODS ENDING NOT EARLIER THAN NOVEMBER 30 AND NOT LATER THAN JANUARY 31 ARE CONSIDERED CALENDAR-YEAR PERIODS. IN THOSE STATES WHERE THE REGISTRATION PERIOD IS DEFINITELY REMOVED FROM THE CALENDAR YEAR, REGISTRATION FIGURES WERE OBTAINED FOR THE CALENDAR-YEAR PERIOD.

2/ WHEREVER POSSIBLE PUBLICLY OWNED VEHICLES AND VEHICLES NOT FOR HIGHWAY USE HAVE BEEN ELIMINATED FROM THESE COLUMNS.

3/ THE BUS FIGURES GIVEN REPRESENT COMMERCIAL BUSSES IN MOST CASES, ALTHOUGH IN SOME STATES CONTRACT SCHOOL BUSSES ARE INCLUDED. FOR STATES WHICH DID NOT SUPPLY SUCH INFORMATION, THE SEGREGATION OF BUSSES FROM OTHER MOTOR VEHICLES HAS BEEN APPROXIMATED FROM OTHER DATA AVAILABLE.

4/ DATA ON FEDERAL VEHICLES OBTAINED THROUGH AGENCY OF PROCUREMENT DIVISION, DEPARTMENT OF THE TREASURY. VEHICLES OWNED BY THE MILITARY SERVICES, INCLUDED IN PRIOR YEARS, ARE OMITTED FROM THESE COLUMNS. FEDERAL VEHICLES NOT ASSIGNED TO PARTICULAR STATES WERE LISTED AS "AT LARGE" IN PREVIOUS TABLES. IN THIS TABLE SUCH VEHICLES HAVE BEEN ASSIGNED TO STATES IN PROPORTION TO THOSE REPORTED BY STATES.

5/ STATE, COUNTY, AND MUNICIPAL VEHICLES ARE INCLUDED WITH PRIVATE AND COMMERCIAL REGISTRATIONS IN COLORADO, KANSAS, MASSACHUSETTS, MICHIGAN, NEW HAMPSHIRE, AND VERMONT. A SMALL NUMBER OF FEDERAL VEHICLES ARE INCLUDED IN THE FIGURES FOR INDIANA, KENTUCKY, LOUISIANA, MONTANA, NEW YORK, PENNSYLVANIA, AND VIRGINIA.

6/ FIGURES FOR TRAILERS AND SEMITRAILERS ARE AS REPORTED. APPARENT INCONSISTENCIES ARE DUE TO THE FACT THAT SOME STATES REQUIRE THE REGISTRATION OF TOURIST TRAILERS, LIGHT WORK TRAILERS, AND SIMILAR VEHICLES, WHEREAS OTHER STATES REGISTER ONLY FREIGHT-CARRYING TRAILERS AND SEMITRAILERS. WHEREVER POSSIBLE PUBLICLY OWNED VEHICLES AND VEHICLES NOT FOR HIGHWAY USE HAVE BEEN ELIMINATED FROM THIS COLUMN.

7/ FIGURES INCLUDE NEW-CAR, USED-CAR, AND MOTORCYCLE DEALERS REGISTRATIONS AND SOME WRECKER AND REPAIRER REGISTRATIONS. DATA ON DEALERS EXTRA PLATES ARE INCOMPLETE, ALTHOUGH THEY ARE INCLUDED WITH DEALERS REGISTRATIONS IN SOME STATES.

8/ TRAILERS OF 1,000 POUNDS CAPACITY OR MORE PROHIBITED ON HIGHWAYS, ALTHOUGH PERMITTED IN CITIES UNDER CITY LICENSES. TRACTOR-SEMITRAILERS REGISTERED AS TRUCKS. LIGHT TRAILERS PERMITTED BUT NOT REGISTERED.

9/ COMMERCIAL FULL TRAILERS INCLUDED WITH TRUCKS.

10/ TAXICABS INCLUDED WITH TRUCKS.

11/ TRAILERS INCLUDED WITH TRUCKS.

12/ TRUCKS UNDER 1,500 POUNDS CAPACITY INCLUDED WITH PASSENGER CARS.

13/ INCLUDES 547 AUTOMOBILES OF THE DIPLOMATIC CORPS.

205764

STATE MOTOR-VEHICLE RECEIPTS - 1941

COMPILED FOR CALENDAR YEAR FROM REPORTS OF STATE AUTHORITIES 1/

TABLE MV-2, 1941
ISSUED JUNE 1942

STATE	TOTAL RECEIPTS	REGISTRATION FEES 2/								MISCELLANEOUS RECEIPTS									STATE
		MOTOR VEHICLES					OTHER VEHICLES		TOTAL	DEALERS LICENSES	OPERATORS AND CHAUFFEURS PERMITS	CERTIFICATE OF TITLE FEES	SPECIAL TITLING TAXES 4/	FINES AND PENALTIES	TRANSFER OR REREGISTRATION FEES	ESTIMATED SERVICE CHARGES, LOCAL COLLECTORS 5/	OTHER RECEIPTS LESS UNCLASSIFIED REFUNDS	TOTAL	
		PASSENGER VEHICLES			TRUCKS AND TRACTOR-TRUCKS	TOTAL	TRAILERS	MOTORCYCLES											
		AUTOMOBILES (INCLUDING TAXICABS)	BUSSES 3/	TOTAL															
	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	
ALABAMA	6,431	4,100	131	4,231	1,317	5,548	74	6	5,628	3	601	-	-	182	-	-	17	803	ALABAMA
ARIZONA	1,352	400	25	425	497	922	103	2	1,027	6	31	93	-	7	17	-	171	325	ARIZONA
ARKANSAS	3,935	2,130	94	2,224	1,135	3,359	199	1	3,559	101	252	-	-	4	15	-	4	376	ARKANSAS
CALIFORNIA 6/	31,927	22,166	(7/)	22,166	5,884	28,050	1,376	91	29,517	98	119	-	-	265	1,691	-	237	2,410	CALIFORNIA 6/
COLORADO	2,953	8/ 1,778	-	1,778	8/ 497	2,275	46	2	2,323	35	168	197	-	57	100	-	73	630	COLORADO
CONNECTICUT	8,249	3,417	158	3,575	1,737	5,312	27	7	5,346	115	2,096	-	-	179	178	-	335	2,903	CONNECTICUT
DELAWARE	1,456	739	(7/)	739	381	1,120	50	1	1,171	9	114	68	-	52	36	-	6	285	DELAWARE
FLORIDA	9,463	5,219	266	5,485	2,398	7,883	417	10	8,310	29	614	314	-	-	1	144	51	1,153	FLORIDA
GEORGIA	2,951	1,238	160	1,398	565	1,963	205	3	2,171	17	725	-	-	-	11	-	27	780	GEORGIA
IDAHO	1,542	673	10	683	543	1,226	76	2	1,304	25	144	57	-	-	-	-	12	238	IDAHO
ILLINOIS	26,567	16,845	(7/)	16,845	7,248	24,093	254	19	24,366	106	601	616	-	28	637	-	213	2,201	ILLINOIS
INDIANA	11,628	7,147	156	7,303	1,896	9,199	433	11	9,643	55	834	408	-	39	480	294	-125	1,985	INDIANA
IOWA	14,231	9,716	(7/)	9,716	3,378	13,094	244	10	13,348	78	675	-	-	-	252	-	-122	883	IOWA
KANSAS	4,743	2,923	(7/)	2,923	1,019	3,942	79	6	4,027	42	392	171	-	67	33	-	11	716	KANSAS
KENTUCKY	6,076	1,960	119	2,079	1,367	3,446	(9/)	3	3,449	26	621	-	1,645	16	299	-	20	2,627	KENTUCKY
LOUISIANA	3,486	10/ 1,087	118	1,205	1,236	2,441	888	5	3,334	9	159	-	-	8	-	-	-24	152	LOUISIANA
MAINE	4,434	2,288	21	2,309	11/ 1,206	3,515	12/ 31	4	3,550	81	570	-	-	35	121	-	77	884	MAINE
MARYLAND	7,830	3,599	221	3,820	743	4,563	194	5	4,762	58	413	286	1,689	498	164	-	-40	3,068	MARYLAND
MASSACHUSETTS	7,905	3,122	163	3,285	1,591	4,876	25	2	4,903	69	2,917	-	-	2	-	-	14	3,002	MASSACHUSETTS
MICHIGAN	27,163	13/ 16,017	(7/)	16,017	13/ 6,306	22,323	1,475	19	23,817	102	1,452	1,092	-	-	434	-	266	3,346	MICHIGAN
MINNESOTA 14/	19,530	14,124	222	14,346	3,851	18,197	680	13	18,890	61	182	-	-	170	-	-	227	640	MINNESOTA 14/
MISSISSIPPI	3,433	1,748	231	1,979	1,264	3,243	26	2	3,271	9	110	-	-	35	-	-	8	162	MISSISSIPPI
MISSOURI	11,713	8,201	(7/)	8,201	1,632	9,833	218	13	10,064	106	496	581	-	4	438	-	24	1,649	MISSOURI
MONTANA	1,797	964	(7/)	964	397	1,361	29	1	1,391	20	189	70	-	64	52	-	11	406	MONTANA
NEBRASKA	3,195	1,366	27	1,393	1,078	2,471	66	2	2,539	42	322	180	-	-	86	-	26	656	NEBRASKA
NEVADA	405	191	1	192	119	311	11	1	323	3	57	-	-	2	14	-	6	82	NEVADA
NEW HAMPSHIRE	3,291	-	-	-	-	2,724	-	4	2,728	23	395	-	-	87	-	-	58	563	NEW HAMPSHIRE
NEW JERSEY	24,349	11,874	316	12,190	4,528	16,718	450	11	17,179	79	4,519	584	-	262	317	-	1,409	7,170	NEW JERSEY
NEW MEXICO	2,213	1,082	68	1,150	632	1,782	70	1	1,853	12	191	40	-	5	-	-	112	360	NEW MEXICO
NEW YORK	53,978	36,207	-	36,207	12,670	48,877	845	48	49,770	210	1,564	-	-	585	1,427	-	422	4,208	NEW YORK
NORTH CAROLINA	9,777	5,462	110	5,572	3,114	8,686	587	8	9,281	42	178	235	-	64	-	-	-23	496	NORTH CAROLINA
NORTH DAKOTA	2,025	1,286	15	1,301	409	1,710	6	1	1,717	22	176	74	34	-	-	-	2	308	NORTH DAKOTA
OHIO	33,421	17,317	(7/)	17,317	9,671	26,988	1,707	48	28,743	216	926	685	-	173	507	2,117	54	4,678	OHIO
OKLAHOMA	7,086	2,820	248	3,068	1,758	4,826	764	6	5,596	55	549	368	-	-	-	400	118	1,490	OKLAHOMA
OREGON	4,416	1,766	43	1,809	15/ 1,485	3,294	(7/)	6	3,300	27	762	259	-	41	-	-	27	1,116	OREGON
PENNSYLVANIA	41,326	21,715	690	22,405	9,507	31,912	714	35	32,661	429	3,577	2,555	-	10	1,803	-	291	8,665	PENNSYLVANIA
RHODE ISLAND	3,425	2,115	52	2,167	580	2,747	6	4	2,757	17	494	-	-	-	112	-	45	668	RHODE ISLAND
SOUTH CAROLINA	2,749	1,070	49	1,119	801	1,920	198	1	2,119	34	503	-	-	39	37	-	17	630	SOUTH CAROLINA
SOUTH DAKOTA	1,879	1,482	5	1,487	277	1,764	48	1	1,813	19	-	31	-	-	-	-	16	66	SOUTH DAKOTA
TENNESSEE	7,077	-	-	-	-	6,123	-	-	6,123	(16/)	644	-	-	161	-	-	149	954	TENNESSEE
TEXAS	26,671	14,570	189	14,759	7,840	22,599	1,077	20	23,696	46	382	639	1,491	-	295	-	122	2,975	TEXAS
UTAH	1,335	601	(7/)	601	17/ 566	1,167	(7/)	1	1,168	16	35	87	-	1	26	-	2	167	UTAH
VERMONT	2,750	18/ 1,613	15	1,628	18/ 685	2,313	48	3	2,364	29	309	-	-	-	63	-	-15	386	VERMONT
VIRGINIA	8,363	5,491	-	5,491	1,738	7,229	209	8	7,446	75	123	404	-	-	255	-	60	917	VIRGINIA
WASHINGTON	5,461	1,569	78	1,647	1,363	3,010	359	7	3,376	17	1,452	181	-	88	133	195	19	2,085	WASHINGTON
WEST VIRGINIA	7,180	4,276	32	4,308	1,220	5,528	82	6	5,616	54	193	195	1,036	-	75	-	11	1,564	WEST VIRGINIA
WISCONSIN	14,862	10,365	179	10,544	3,092	13,636	517	16	14,169	36	355	8	-	-	283	-	11	693	WISCONSIN
WYOMING	652	345	-	345	238	583	55	-	638	8	-	-	-	-	4	-	2	14	WYOMING
DISTRICT OF COLUMBIA	1,985	882	20	902	464	1,366	21	3	1,390	21	326	97	-	-	34	-	117	595	DISTRICT OF COLUMBIA
PARTIAL TOTALS 19/	-	277,066	4,232	281,298	111,923	-	14,989	479	-	-	-	-	-	-	-	-	-	-	PARTIAL TOTALS 19/
FULL TOTALS	490,666	-	-	-	-	402,068	-	-	417,536	2,792	32,507	10,575	5,895	3,230	10,430	3,150	4,551	73,130	FULL TOTALS

1/ RECEIPTS FOR REGISTRATION PERIODS ENDING NOT EARLIER THAN NOVEMBER 30 AND NOT LATER THAN JANUARY 31 ARE CONSIDERED CALENDAR-YEAR RECEIPTS. IN THOSE STATES WHERE THE REGISTRATION PERIOD IS DEFINITELY REMOVED FROM THE CALENDAR YEAR, REGISTRATION RECEIPTS WERE OBTAINED FOR THE CALENDAR-YEAR PERIOD.

2/ SEGREGATION OF REGISTRATION FEES BY TYPE OF VEHICLE NOT AVAILABLE FOR NEW HAMPSHIRE AND TENNESSEE. TOTAL MOTOR-VEHICLE REGISTRATION FEES INCLUDE TRAILER FEES IN NEW HAMPSHIRE, AND TRAILER, MOTORCYCLE, AND DEALERS LICENSE FEES IN TENNESSEE.

3/ WHERE NO FEES ARE TABULATED THE FEES OF BUSSES ARE INCLUDED WITH THOSE OF AUTOMOBILES, UNLESS OTHERWISE NOTED.

4/ SPECIAL TITLE TAXES IMPOSED UNDER GENERAL SALES TAX LEVIES ARE NOT INCLUDED.

5/ IN MANY STATES COUNTY OR LOCAL OFFICERS ARE ALLOWED SERVICE CHARGES FOR ISSUING REGISTRATIONS, OPERATORS LICENSES, ETC. IN THE MAJORITY OF CASES THESE CHARGES ARE INCLUDED IN REGISTRATION AND OTHER FEES. THE AMOUNTS SHOWN IN THIS COLUMN ARE ESTIMATES OF SERVICE CHARGES COLLECTED AND RETAINED BY LOCAL OFFICIALS AND NOT REPORTED ELSEWHERE IN THE TABLE.

6/ REGISTRATION FEES INCLUDE PROCEEDS OF STATE "VEHICLE LICENSE FEES", $15,757,038, IMPOSED IN ADDITION TO THE REGULAR REGISTRATION FEES OF $13,760,350.

7/ INCLUDED WITH TRUCK FEES.

8/ FEES OF LIGHT TRUCKS INCLUDED WITH THOSE OF AUTOMOBILES.

9/ TRAILERS OF 1,000 POUNDS CAPACITY OR MORE PROHIBITED ON HIGHWAYS, ALTHOUGH PERMITTED IN CITIES UNDER CITY LICENSES. TRACTOR-SEMITRAILERS REGISTERED AS TRUCKS. LIGHT TRAILERS PERMITTED BUT NOT REGISTERED.

10/ DECREASE FROM 1940 RECEIPTS WAS DUE TO REDUCTION OF AUTOMOBILE REGISTRATION FEE TO THREE DOLLARS.

11/ INCLUDES FEES OF COMMERCIAL FULL TRAILERS.

12/ FEES OF LIGHT TRAILERS AND COMMERCIAL SEMITRAILERS ONLY. FEES OF COMMERCIAL FULL TRAILERS INCLUDED WITH THOSE OF TRUCKS.

13/ TAXICAB FEES INCLUDED WITH THOSE OF TRUCKS.

14/ LARGE INCREASE IN 1941 REGISTRATION RECEIPTS WAS DUE TO INCLUSION OF 1942 FEES COLLECTED IN LATTER PART OF 1941 BECAUSE OF CHANGE IN LAW.

15/ INCLUDES TRAILER FEES.

16/ INCLUDED WITH REGISTRATION FEES.

17/ INCLUDES FEES OF BUSSES AND TRAILERS.

18/ FEES OF TRUCKS UNDER 1,500 POUNDS CAPACITY INCLUDED WITH THOSE OF PASSENGER CARS.

19/ TOTALS OF COLUMNS FOR WHICH FULLY CLASSIFIED DATA WERE NOT AVAILABLE FOR ALL STATES.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF JULY 31, 1942

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama				$4,261,349	$2,315,384	115.2	$198,900	$99,450	0.3	$2,565,784
Arizona				665,314	552,164	28.2	487,374	301,290	4.8	1,515,952
Arkansas				1,275,742	635,815	59.9	810,614	589,082	16.9	1,172,187
California	$281,700	$165,500	11.2	4,473,982	3,124,165	42.8	2,604,776	1,858,813	75.5	2,226,638
Colorado	79,600	79,600		3,541,754	2,270,652	155.4	938,023	529,825	30.9	1,954,086
Connecticut				2,254,409	1,090,642	27.8	1,186,822	774,873	6.7	424,612
Delaware				428,810	210,460	9.0	268,040	134,020	8.4	1,513,348
Florida	65,718	45,075	.3	3,419,614	2,017,687	52.7	150,000	112,500	.9	2,909,335
Georgia	193,521	96,760	10.9	8,062,690	4,254,197	282.7	3,030,687	1,584,005	107.6	6,189,049
Idaho	307,561	218,529	17.3	1,109,663	812,177	57.3	425,377	327,494	10.6	1,529,251
Illinois	739,050	377,875	7.7	7,008,025	4,078,892	114.5	2,524,528	1,602,596	23.1	5,291,261
Indiana	805,800	402,900	15.5	6,425,689	3,129,737	89.7				2,907,857
Iowa	418,768	196,300	21.3	3,644,785	1,531,495	92.7	123,176	27,200	4.6	2,309,213
Kansas	346,856	181,060	19.0	5,964,404	3,264,041	240.2	659,063	339,177	34.9	4,550,299
Kentucky	283,274	141,637	6.2	5,089,499	2,577,107	87.0	3,263,364	1,948,234	31.5	570,843
Louisiana	15,648	11,736		1,159,867	798,961	22.4	3,755,006	2,111,722	67.6	3,423,906
Maine	155,480	77,740	2.9	1,789,292	977,337	19.2	78,610	39,305	.1	1,006,859
Maryland				2,253,781	1,102,489	12.0	810,000	573,625	4.3	780,082
Massachusetts	444,074	221,658	5.5	1,493,894	821,486	8.2	1,170,360	581,593	8.4	3,802,058
Michigan	28,000	21,000	.4	3,526,516	2,343,458	73.7	1,477,100	1,006,775	61.0	2,349,634
Minnesota	934,753	576,391	44.1	8,463,111	4,597,110	347.1	499,190	249,595	9.3	2,644,051
Mississippi				2,923,908	1,466,504	167.4	331,300	248,475	5.3	2,077,457
Missouri	699,456	349,728	14.3	9,323,372	5,599,469	158.2	2,044,687	1,057,614	21.8	3,263,308
Montana	680,704	409,067	18.3	3,076,532	2,018,054	158.3	238,580	158,250	25.7	4,088,704
Nebraska	687,804	358,622	57.6	2,260,130	1,143,999	128.5	1,167,560	874,778	76.6	3,644,730
Nevada	97,818	85,002	6.4	1,479,326	1,324,000	40.8	544,729	486,949	13.9	413,339
New Hampshire				1,682,125	1,131,880	19.4				683,756
New Jersey				85,262	42,631		758,830	551,250	2.2	2,381,851
New Mexico	55,592	38,583	11.0	726,233	566,823	26.9	818,715	671,755	58.5	1,977,719
New York	804,019	514,788	9.3	7,952,443	5,067,685	77.0	429,000	193,500	6.4	5,204,953
North Carolina	868,374	455,347	24.2	1,122,567	599,147	54.5	882,016	443,952	10.0	3,484,755
North Dakota	338,180	215,387	41.9	2,922,673	1,757,395	190.0	2,841,800	1,636,982	181.8	3,010,930
Ohio	2,757,360	1,755,944	16.2	12,047,192	7,049,032	70.2	3,527,610	1,938,966	27.7	757,003
Oklahoma				2,717,873	1,521,583	57.2	1,787,060	1,090,044	45.0	5,758,946
Oregon	112,243	51,000	10.4	3,755,534	2,128,876	75.7	397,962	318,810	26.2	930,823
Pennsylvania	365,287	182,489	4.3	8,491,988	4,527,019	61.0	3,036,620	2,235,739	18.6	2,667,949
Rhode Island	14,721	7,312		1,032,274	662,318	5.1	479,631	239,757	2.6	602,219
South Carolina				3,279,041	2,073,216	71.1	1,133,990	781,625	19.5	901,776
South Dakota	577,100	414,170	57.5	3,775,133	2,491,345	444.0	615,240	365,050	70.4	2,867,481
Tennessee	326,308	163,154	1.5	4,250,726	2,496,930	95.3				3,485,842
Texas	1,431,060	715,100	51.6	7,197,714	3,399,398	189.4	1,330,145	651,830	40.9	8,632,202
Utah	577,845	434,655	14.8	1,132,810	901,211	26.8	874,733	476,706	23.2	766,837
Vermont	237,535	107,173	2.8	955,791	608,067	18.0	36,906	18,453	.3	396,743
Virginia	305,788	153,839	8.8	2,963,766	1,625,853	42.2	35,490	17,745	.4	2,757,658
Washington	40,957	27,150	.8	2,639,710	1,473,993	30.1	211,457	124,500	3.4	1,885,522
West Virginia	423,506	208,401	4.4	2,485,653	1,502,080	24.4	884,359	488,979	8.8	1,059,368
Wisconsin	1,289,778	794,390	51.9	5,010,079	3,311,467	117.5	352,531	251,350	10.4	3,369,384
Wyoming	491,888	353,584	27.4	1,358,229	1,040,201	104.7	140,715	115,749	12.1	1,069,972
District of Columbia				353,629	206,688	.9	1,008,052	593,356	3.6	28,530
Hawaii	3,444	3,444		1,236,596	951,269	11.8	476,174	456,702	4.7	1,306,261
Puerto Rico	166,075	81,840	3.1	2,425,620	1,199,666	17.3	37,725	27,994	.2	527,567
TOTALS	18,452,645	10,693,930	600.8	177,026,119	102,393,305	4,421.4	50,935,127	31,308,034	1,227.6	121,639,890

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF JULY 31, 1942

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama				$562,252	$300,210	10.9	$23,031	$11,050		$590,870
Arizona	$63,345	$46,046	2.9	142,693	103,688	6.7	69,383	49,650		414,777
Arkansas				662,760	329,780	32.4	135,656	67,416	8.0	180,507
California	4,995	4,995		703,607	555,721	8.8	18,050	9,500		954,372
Colorado				137,809	78,621	3.2	152,387	35,323	5.0	497,995
Connecticut				275,575	120,602	4.8				194,940
Delaware	43,350	21,675	5.0	8,040	4,020		102,873	37,618	3.9	251,159
Florida	108,248	54,124	.2	188,120	103,530	6.0				399,766
Georgia				1,326,307	775,503	82.8	604,762	302,381	36.6	938,371
Idaho				391,287	259,816	12.2	38,149	23,587	2.8	183,196
Illinois	40,000	20,000	2.2	1,263,384	631,692	63.2	219,000	109,500	3.7	771,146
Indiana	80,000	40,000	5.3	1,269,466	599,282	48.8				927,827
Iowa	109,341	51,500	20.6	543,562	242,570	79.1	32,056	15,050	8.8	585,167
Kansas	26,060	17,170		1,808,040	904,857	97.9	177,337	88,669	23.4	1,039,534
Kentucky	140,794	22,951	4.5	1,041,841	378,281	29.9	35,726	6,400	.1	319,467
Louisiana				15,460	7,730		674,013	267,134	15.3	533,140
Maine	64,770	32,385	3.8	116,796	58,398	4.1	39,383	19,614	.5	142,227
Maryland				226,410	113,205	1.8				345,564
Massachusetts	241,682	128,841	3.4	462,552	264,842	6.7				522,088
Michigan	51,800	25,900		672,898	336,449	29.4	219,670	109,835	.2	655,791
Minnesota	34,062	17,031	.1	1,031,533	521,505	98.0	298,260	148,730	19.6	560,306
Mississippi				1,504,188	714,285	59.0				388,361
Missouri	162,104	81,052	12.5	560,911	271,114	57.7	186,244	63,193	32.4	1,030,471
Montana	39,842	22,609	7.3	252,900	148,113	23.7	13,569	7,715	4.5	904,396
Nebraska	54,055	29,603	10.8	131,727	68,477	13.9				695,483
Nevada				88,578	56,842	4.6	107,839	90,444	11.9	144,428
New Hampshire				241,995	164,766	3.6				87,696
New Jersey	278,480	149,240	6.2	163,825	100,815	7.6				561,260
New Mexico				262,325	169,211	19.5				250,178
New York				1,431,250	756,263	15.9				1,076,253
North Carolina	240,526	120,230	17.3	394,121	214,458	20.8	69,820	20,000	5.0	638,786
North Dakota				7,382	7,382		808,050	793,860	42.7	755,172
Ohio				901,610	498,775	14.5	236,950	118,475	5.0	1,303,467
Oklahoma				120,588	63,680	4.0	1,215,706	641,924	71.9	860,552
Oregon	108,767	66,350	3.5	286,801	163,737	16.8	30,482	18,000	1.3	333,502
Pennsylvania				292,174	146,052	5.5	194,147	97,051	2.5	724,357
Rhode Island	2,207	1,096		78,192	42,846	.8	33,245	16,622	.2	80,038
South Carolina	202,900	65,124	.5	24,944	18,355					394,792
South Dakota				9,056	9,056		1,143,430	1,047,600	114.5	757,118
Tennessee	295,522	147,761	8.4	846,000	433,000	34.8	158,042	79,021	4.6	749,292
Texas				360,490	177,895	26.5				2,241,944
Utah	55,085	27,543	2.5	99,045	73,878	1.5				305,370
Vermont				222,515	117,328	8.9				53,580
Virginia	322,312	142,308	7.7	49,604	25,572	.7				679,170
Washington				337,818	135,362	7.8	786,758	409,402	64.8	1,029
West Virginia	175,950	87,975	5.3	115,470	57,122	1.1				511,059
Wisconsin	129,993	61,830	.3	1,157,387	528,605	42.4				581,141
Wyoming				464,690	204,982	31.7				214,778
District of Columbia							107,026	52,000	.9	111,600
Hawaii				1,279	1,279		49,687	49,687	.4	285,187
Puerto Rico				74,942	40,110	3.1				216,668
TOTALS	3,076,190	1,485,339	130.3	23,332,199	12,099,662	1,053.1	7,980,731	4,806,451	490.5	27,945,338

U. S. GOVERNMENT PRINTING OFFICE : 1942

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Report of the Chief of the Bureau of Public Roads, 1932. 5 cents.
Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.
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Indexes to PUBLIC ROADS, volumes 6–8 and 10–21, inclusive.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

REPORTS IN COOPERATION WITH UNIVERSITY OF ILLINOIS

No. 303. . . Solutions for Certain Rectangular Slabs Continuous Over Flexible Support.
No. 304. . . A Distribution Procedure for the Analysis of Slabs Continuous Over Flexible Beams.
No. 313. . . Tests of Plaster-Model Slabs Subjected to Concentrated Loads.
No. 314. . . Tests of Reinforced Concrete Slabs Subjected to Concentrated Loads.
No. 315. . . Moments in Simple Span Bridge Slabs With Stiffened Edges.

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.
Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.
Act III.—Uniform Motor Vehicle Civil Liability Act.
Act IV.—Uniform Motor Vehicle Safety Responsibility Act.
Act V.—Uniform Act Regulating Traffic on Highways.
Model Traffic Ordinances.

A complete list of the publications of the Public Roads Administration, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to Public Roads Administration, Willard Bldg., Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF JULY 31, 1942

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$19,100	$19,100			1	$504,525	$502,903	7	2	1	$52,335	$52,335	2		5	$908,129
Arizona						138,529	129,838		1	2	4,095	4,095			1	230,684
Arkansas	9,364	9,364			3	171,586	169,876	1		1	14,360	13,964			5	643,828
California						688,453	688,453	3			3,775	3,775			1	2,308,368
Colorado						612,609	612,609	5		2	11,111	11,111			5	741,933
Connecticut						8,652	8,652				231,374	222,740		1		532,031
Delaware						191,599	189,867	1	1		508,406	321,785	2			179,564
Florida						843,067	840,887	8		6	78,493	78,433			13	871,496
Georgia	160,690	160,690	2			665,593	665,593	3	5	9	785,985	785,985	2	4	9	1,504,838
Idaho						185,563	176,891	2			4,189	4,189			2	448,975
Illinois	505,480	425,230	1	1	1	957,927	954,981	6		2	384,599	384,399	1	1	29	2,549,863
Indiana						440,361	440,361	2	1	13	85,861	85,861			26	1,160,246
Iowa	142,887	137,775	1	1	9	1,401,276	1,148,026	10	1	16	79,765	78,845			21	638,171
Kansas	196,324	196,119	2		2	616,568	616,568	9		3	82,621	82,621	1		5	1,294,058
Kentucky						408,223	408,223	4			23,850	23,850	1			396,031
Louisiana	252,494	252,494	2			338,486	338,486	6			658,006	655,120	5		1	736,600
Maine						253,884	253,884	1		1						284,591
Maryland	7,150	7,150			1	884,808	884,330	4		2	30,175	30,175			6	242,604
Massachusetts	84,036	83,820	1			608,552	607,941	3	1		763,830	763,830	2	1		1,250,593
Michigan						1,623,692	1,623,692	4		1	36,245	36,245			10	129,825
Minnesota						943,112	943,112	5	4		4,739	4,739			1	1,241,354
Mississippi	244,800	244,800	4			610,828	610,828	5								641,634
Missouri	13,356	13,356				1,574,876	1,297,892	5	4		20,778	20,778	1			1,462,419
Montana	5,963	5,950				93,828	93,828	1								734,621
Nebraska	96,908	96,836	1		1	410,892	410,892	8			11,853	11,853			2	480,668
Nevada						56,484	56,484	2			22,635	22,635			7	196,853
New Hampshire						76,060	75,735	2								313,962
New Jersey						632,648	507,098	2	2		299,170	239,745	1	1		922,538
New Mexico						71,000	71,000	1			259,103	252,068	3		1	519,348
New York	568,500	566,900	2	1		1,355,198	1,315,859	1	6		502,645	464,285		1		3,726,559
North Carolina	78,355	78,355		1		99,847	99,847	1		2	26,315	26,315			6	1,411,984
North Dakota	216,790	216,790	3			192,655	192,655	1								872,117
Ohio						2,752,053	2,428,331	10		1	691,590	487,460	2		4	1,428,559
Oklahoma						888,013	886,093	6		3	392,744	354,707	3		3	1,435,846
Oregon						603,387	476,181	3			13,433	13,433			4	160,571
Pennsylvania						3,160,659	3,146,130	15			235,355	235,355	1			2,506,669
Rhode Island	1,472	1,462				2,193	2,193									273,921
South Carolina	9,420	9,420			2	47,842	47,842	1	1		235,073	119,940		3	6	975,666
South Dakota						515,821	499,871	9			41,200	41,200	2			830,897
Tennessee						1,256,182	1,256,182	6	1		430,180	430,180	1	2	3	672,789
Texas	138,448	136,041	1		1	795,830	779,154	6		2						2,388,145
Utah						42,917	42,917	1		1	63,531	63,531			23	362,966
Vermont						324,331	294,552	2								113,888
Virginia						352,754	332,794	3	1	1						965,312
Washington						1,041,232	344,558	5			2,478	2,478			1	447,340
West Virginia	172,050	172,050	3			834,123	834,123	4		2	3,330	3,330			1	324,320
Wisconsin	38,476	38,381		1	1	381,214	380,470	3		4	12,687	12,687			2	1,671,665
Wyoming						6,210	6,210			4	6,159	6,158			2	428,049
District of Columbia						285,934	285,934		1							91,161
Hawaii						211,977	211,462	2								282,756
Puerto Rico						780,618	772,706	11								363,914
TOTALS	2,962,063	2,872,083	23	5	22	31,944,671	29,964,994	200	32	79	7,114,073	6,452,235	30	14	205	45,300,919

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

FEDERAL WORKS AGENCY
PUBLIC ROADS ADMINISTRATION

VOL. 23, NO. 6 ▽ ▽ OCTOBER NOVEMBER-DECEMBER 1942

BLACKOUT HEADLAMP ILLUMINATES ONLY LOW SIGNS AND MARKINGS

For sale by the Superintendent of Documents, Washington, D. C. - - - - - - - - - - - - - - - - See page 2 of cover for prices

PUBLIC ROADS

▸ ▸ ▸ *A Journal of Highway Research*

issued by the

FEDERAL WORKS AGENCY
PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 23, No. 6

October–November–December 1942

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Commissioner of Public Roads, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

HIGHWAY SIGNS AND MARKINGS FOR BLACKOUT CONDITIONS

BY THE DIVISION OF TRAFFIC AND SAFETY, OHIO DEPARTMENT OF HIGHWAYS AND THE DIVISION OF HIGHWAY TRANSPORT, PUBLIC ROADS ADMINISTRATION

Reported by **GEORGE J. FISHER**, Assistant Traffic Engineer, Ohio Department of Highways, and **O. K. NORMANN**, Highway Engineer-Economist, Public Roads Administration

Marking important highways so that essential vehicular traffic can move with comparative safety during blackouts may become one of the most critical traffic-control problems during the present emergency.

This report includes results of a study conducted during blackout conditions by driving passenger cars equipped with lights approved by the War Department over highways on which a large variety of standard and special signs and markings were installed.

A highway, either urban or rural, can be marked so that vehicles equipped with the approved blackout lighting system may be operated with comparative safety at speeds up to 20 miles per hour.

The need for most types of signs will be reduced during blackouts while the need for marking centerlines and hazardous objects within the roadway will be increased.

Signs, to be effective, must be either illuminated or reflectorized and mounted with their legends within 18 inches of the roadway elevation and as near the roadway as practicable. Pavement markings, to be effective, must be reflectorized.

THE complete elimination of all lights that would be visible at night to enemy aircraft has thus far been ordered by military authorities in very few sections of the United States and then only for relatively short periods. However, test blackouts to train Civilian Defense organizations and to make the necessary preparations in case frequent blackouts become necessary have been conducted in all sections of the country, especially in the coastal areas. While it may never be necessary to have blackouts over extended areas of the United States for long periods, present conditions warrant taking the necessary steps to insure that problems arising in blackouts can be handled quickly and efficiently.

One of the problems involved is the movement of essential vehicular traffic. Even during blackouts lasting for only short periods, certain military vehicles and emergency civilian vehicles must be in operation. In any area having frequent blackouts, it can also be assumed that vehicles carrying materials and workers to and from war production plants will be permitted to operate. Otherwise, production would be seriously curtailed.

Regulations governing traffic movement during blackouts, issued by the Office of Civilian Defense[1] and based on standards set by the War Department, specify that only vehicles equipped with blackout lights approved by the War Department will be permitted to operate.

The approved blackout lighting system for civilian vehicles, decided upon after extensive tests by the Blackout and Traffic Control Branch of the Engineer Board of the War Department, consists of one head lamp, two front clearance lamps, and a combination stop and tail lamp (fig. 1). The head lamp or driving lamp emits a beam that has only a small fraction of the intensity of an ordinary seal-beam headlight. It must be mounted as near the front of the vehicle as possible, between the left side and center of the vehicle and at a height of not less than 36 or more than 55 inches above the road surface. When properly adjusted, the visual cut-off of the top of the beam slopes down at the rate of 2 to 3 inches every 10 feet and illuminates a level road surface uniformly for 20 to 100 feet in front of the car. Beyond 100 feet, the illumination on the road surface gradually decreases until there are no light rays falling beyond 200 feet from the vehicle.

The two clearance lamps are mounted on the front fenders as far apart and as near the normal headlight height as possible. When viewed from less than 120 feet in front of the vehicle, two separate illuminated areas are visible in each lamp. Beyond this distance, the two separate illuminated areas in each lamp appear as a single light.

The combination stop and tail lamp must be mounted as near the left side as practicable and at a height of not less than 20 inches or more than 50 inches above the road. Only a single area is illuminated when the stop light is turned on, whereas four separate areas are illuminated in the tail lamp. The four individual areas can be seen up to a distance of 60 feet. Between 60 and 120 feet, the illuminated portion appears as two separate areas, while at distances over 120 feet the areas appear to merge and only one light is seen. This design feature is of assistance to a driver in estimating the distance to another vehicle when only the taillight is visible.

Lights used on the vehicle during normal conditions, including dash lights, are turned off when the switch for the blackout lights is turned to the ON position.

Recognizing the need of special devices to regulate, warn, and guide drivers of vehicles during blackouts, the Division of Traffic and Safety of the Ohio Department of Highways in cooperation with the Public Roads Administration conducted this study of the effectiveness of various highway signs and markings

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[1] Blackout Requirements for Highway Movement, prepared under the direction of the Chief of Engineers, United States Army, by the Engineer Board, and published by United States Office of Civilian Defense.

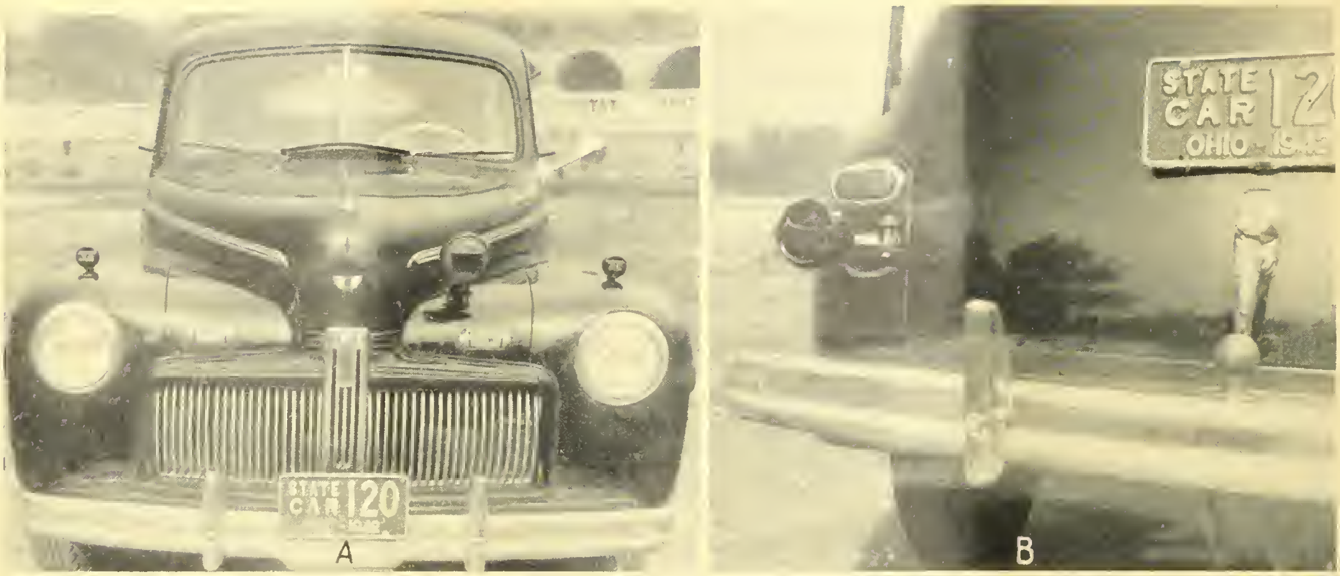


Figure 1.—Automobile Equipped With Approved Blackout Head Lamp, Marker Lamps, and Combination Stop and Tail Lamp.

at the request of the Joint Committee on Uniform Traffic Control Devices.[2] These tests were designed to extend the comprehensive research already conducted by the Engineer Board of the War Department to conditions pertaining particularly to civilian driving.

TESTS CONDUCTED DURING BLACKOUT CONDITIONS

Valuable assistance in the conduct of the study was supplied by representatives of the War Department, the Office of Civilian Defense, and the Lamp Department of the General Electric Engineering Research Laboratories at Nela Park near Cleveland, Ohio. Lights approved by the War Department for use during blackouts were furnished by three manufacturers of these units[3] and installed on passenger cars of the State Highway Patrol, State Highway Department, and Public Roads Administration.

In order to determine the effectiveness of various traffic signs and markings, it was essential that this study be conducted during actual blackout conditions on highways having various surface widths, alinements, and a variety of other design features. A 3.5-mile section of U. S. Route 40 and a 6.2-mile section of State Route 79, both extending through the village of Hebron, Ohio, were selected as the highways along which the traffic control devices included in this study were located.

In addition to 27 existing signs and route markers, 12 standard signs, 45 special test signs, 8 interior-illuminated signs, 6 floodlighted signs, and 20 symbols were installed along the selected route. Outside of the limits of Hebron, 18 different center- and edge-line designs were used on 30 sections of highway, each at least 1,000 feet long, and delineators having plastic buttons as the reflecting units were installed along the outside of 4 curves.

Within the limits of Hebron, a large variety of pavement, curb, and object markings were used and the traffic signal at the intersection of routes 40 and 79 was equipped with a 27-volt transformer. Along the entire route, culvert and bridge headwalls, guardrails, and objects so near the roadway as to cause a possible traffic hazard were marked, employing a large variety of designs and materials such as plain paint, beaded paint, reflector buttons, and materials with reflectorized coatings.

All of the four barricades at the ends of the test sections were also marked to increase their visibility. One was reflectorized. At each of two barricades three red lanterns were used, and at the other barricade two lanterns meeting War Department specification[4] for blackout lighting were used. Four painted or reflectorized boards 10 inches by 10 feet and 33 truncated cones 8 inches in diameter and 6 inches high were used near the barricades to outline the area available for turning the vehicles. Figures 2 and 3 show the layout of the test course.

The first observations of the signs and markings under blackout conditions were made in the early morning hours on Monday, August 3, 1942. Five cars equipped with blackout lights and carrying State highway department personnel that were to act as guides and recorders during the succeeding tests were driven over the 19.4-mile test course (both directions over the 9.7 miles of highway). Traffic had been detoured from the test course by placing the necessary detour signs and stationing a standard barricade and two men from the State highway maintenance organization far enough beyond the barricades at the end of the test section to prevent the drivers of the test vehicles from seeing any normal headlights.

Immediately before the blackout signal sounded, the air-raid wardens of the Hebron Civilian Defense Organization went into action and saw that all street, house, and other lights in the neighborhood of the test route were extinguished or blacked out. State highway patrolmen kept traffic originating within the test area off the rural portions of the route and emergency police of the Civilian Defense Organization were stationed within Hebron to warn pedestrians and to control traffic. They were equipped with blackout flashlights, reflectorized leggings, and 3-foot broomsticks wrapped with reflectorized material.

---

[2] This committee consists of a total of 21 members appointed by the American Association of State Highway Officials, the Institute of Traffic Engineers, and the National Conference on Street and Highway Safety. It is charged with the responsibility of revising the "Manual on Uniform Traffic Control Devices" to cover war emergency conditions.

[3] Eleven sets of blackout lights were furnished, consisting of three sets by the Guide Lamp Division, General Motors Corporation, Anderson, Indiana; four sets by the Corcoran-Brown Lamp Company, Cincinnati, Ohio; and four sets by the C. M. Hall Lamp Company, of Detroit, Michigan.

[4] Blackout Flashlights, Lanterns, and Flares. War Department Specification.

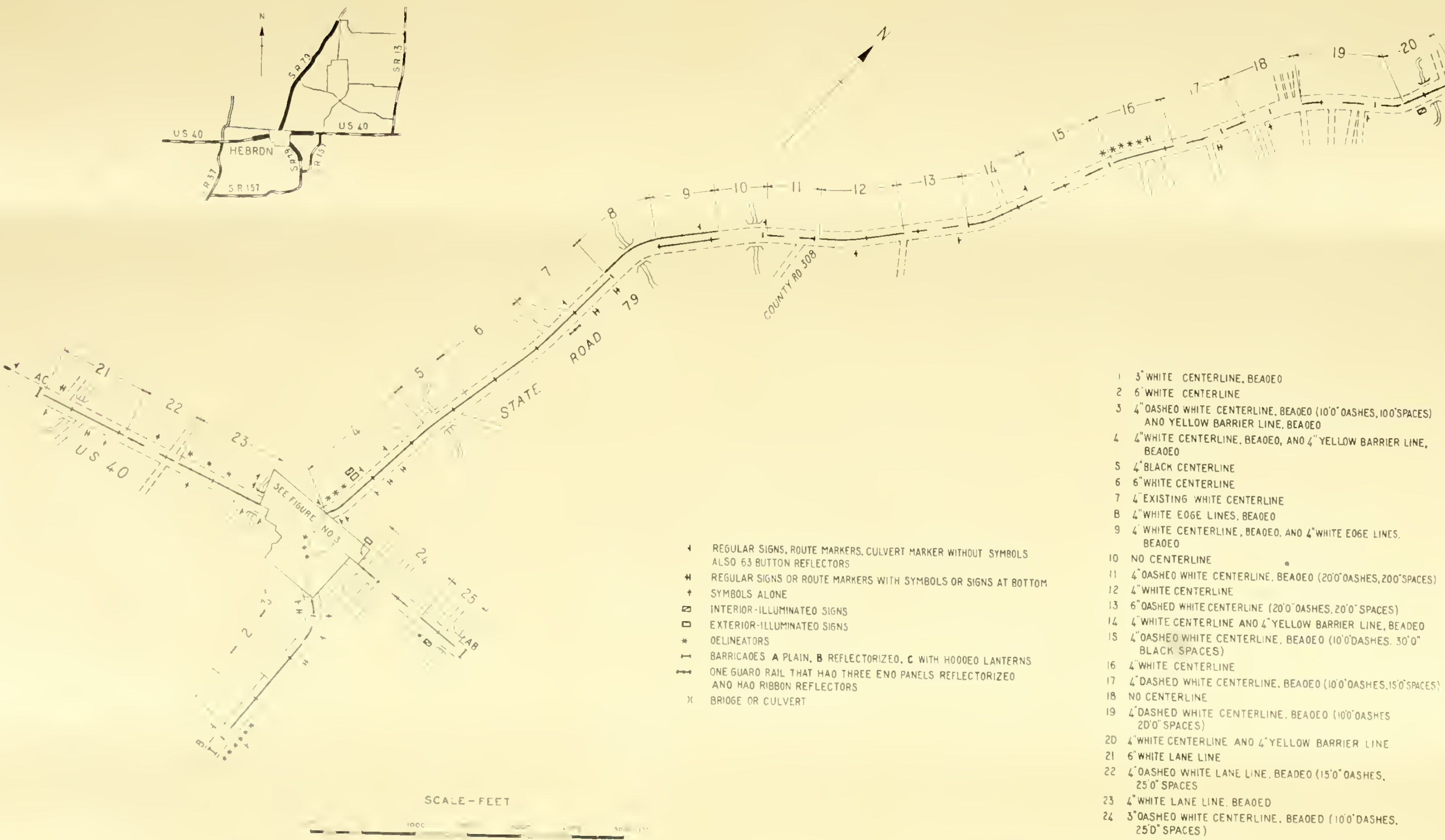

N
S.R. 73
S.R. 13
US 40
US 40
HEBRON
S.R. 79
R 13
R 37
S.R. 157
N
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
AC
AC
AB
B
B
US 40
STATE ROAD 79
COUNTY RD 308
SEE FIGURE NO. 3
SCALE - FEET
1000
REGULAR SIGNS, ROUTE MARKERS, CULVERT MARKER WITHOUT SYMBOLS ALSO 63 BUTTON REFLECTORS
REGULAR SIGNS OR ROUTE MARKERS WITH SYMBOLS OR SIGNS AT BOTTOM
SYMBOLS ALONE
INTERIOR-ILLUMINATED SIGNS
EXTERIOR-ILLUMINATED SIGNS
DELINEATORS
BARRICADES A PLAIN, B REFLECTORIZED, C WITH HOODED LANTERNS
ONE GUARD RAIL THAT HAD THREE END PANELS REFLECTORIZED AND HAD RIBBON REFLECTORS
BRIDGE OR CULVERT
1 3" WHITE CENTERLINE, BEADED
2 6" WHITE CENTERLINE
3 4" DASHED WHITE CENTERLINE, BEADED (10'0" DASHES, 10'0" SPACES) AND YELLOW BARRIER LINE, BEADED
4 4" WHITE CENTERLINE, BEADED, AND 4" YELLOW BARRIER LINE, BEADED
5 4" BLACK CENTERLINE
6 6" WHITE CENTERLINE
7 4" EXISTING WHITE CENTERLINE
8 4" WHITE EDGE LINES, BEADED
9 4" WHITE CENTERLINE, BEADED, AND 4" WHITE EDGE LINES, BEADED
10 NO CENTERLINE
11 4" DASHED WHITE CENTERLINE, BEADED (20'0" DASHES, 20'0" SPACES)
12 4" WHITE CENTERLINE
13 6" DASHED WHITE CENTERLINE (20'0" DASHES, 20'0" SPACES)
14 4" WHITE CENTERLINE AND 4" YELLOW BARRIER LINE, BEADED
15 4" DASHED WHITE CENTERLINE, BEADED (10'0" DASHES, 30'0" BLACK SPACES)
16 4" WHITE CENTERLINE
17 4" DASHED WHITE CENTERLINE, BEADED (10'0" DASHES, 15'0" SPACES)
18 NO CENTERLINE
19 4" DASHED WHITE CENTERLINE, BEADED (10'0" DASHES 20'0" SPACES)
20 4" WHITE CENTERLINE AND 4" YELLOW BARRIER LINE
21 6" WHITE LANE LINE
22 4" DASHED WHITE LANE LINE, BEADED (15'0" DASHES, 25'0" SPACES
23 4" WHITE LANE LINE, BEADED
24 3" DASHED WHITE CENTERLINE, BEADED (10'0" DASHES, 25'0" SPACES)
25 4" WHITE CENTERLINE

FIGURE 2.—CENTERLINES, PAVEMENT EDGE LINES AND SPECIAL MARKERS ON RURAL SECTIONS OF TEST ROUTE.

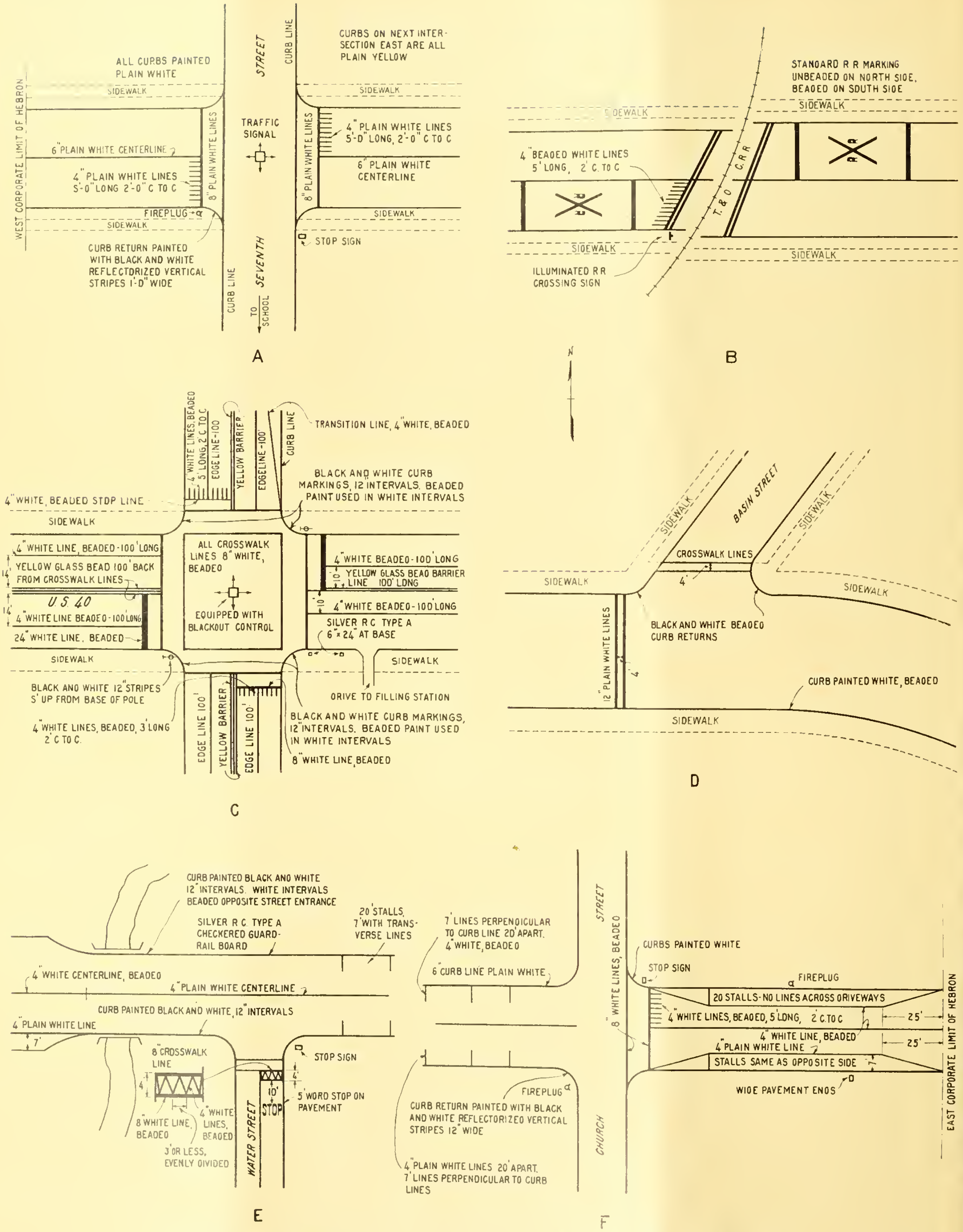
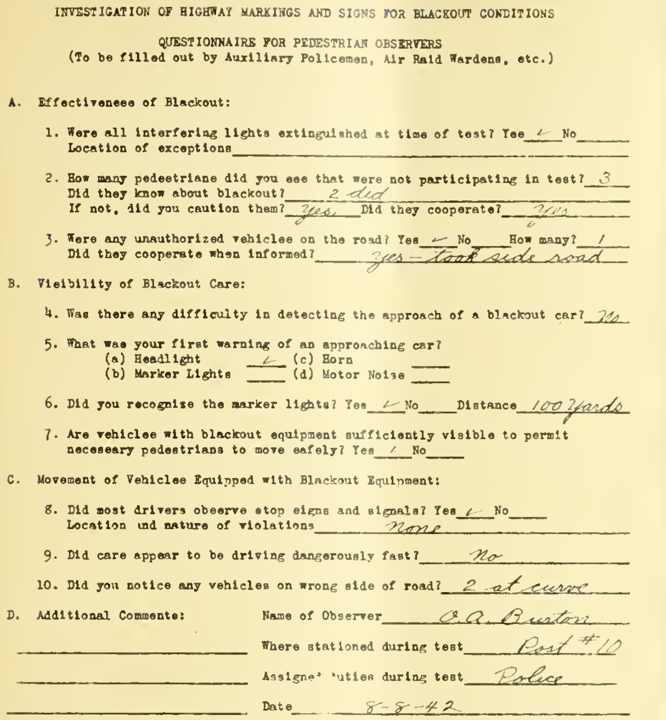

ALL CURBS PAINTED PLAIN WHITE
SIDEWALK
WEST CORPORATE LIMIT OF HEBRON
6" PLAIN WHITE CENTERLINE
4" PLAIN WHITE LINES 5'-0" LONG 2'-0" C TO C
8" PLAIN WHITE LINES
FIREPLUG
CURB RETURN PAINTED WITH BLACK AND WHITE REFLECTORIZED VERTICAL STRIPES 1'-0" WIDE
STREET
SEVENTH
CURB LINE
TO SCHOOL
TRAFFIC SIGNAL
CURBS ON NEXT INTERSECTION EAST ARE ALL PLAIN YELLOW
4" PLAIN WHITE LINES 5'-0" LONG, 2'-0" C TO C
6" PLAIN WHITE CENTERLINE
STOP SIGN
A
STANDARD R R MARKING UNBEADED ON NORTH SIDE, BEADED ON SOUTH SIDE
SIDEWALK
4" BEADED WHITE LINES 5' LONG, 2' C. TO C
T. & O. C. R R
R R
ILLUMINATED R R CROSSING SIGN
B
N
4" WHITE LINES, BEADED 5' LONG, 2' C TO C
EDGE LINE-100'
YELLOW BARRIER
EDGELINE-100'
CURB LINE
TRANSITION LINE, 4" WHITE, BEADED
BLACK AND WHITE CURB MARKINGS, 12" INTERVALS. BEADED PAINT USED IN WHITE INTERVALS
4" WHITE, BEADED STOP LINE
SIDEWALK
4" WHITE LINE, BEADED-100' LONG
YELLOW GLASS BEAD 100' BACK FROM CROSSWALK LINES
14'
U.S. 40
4" WHITE LINE BEADED-100' LONG
24" WHITE LINE, BEADED
ALL CROSSWALK LINES 8" WHITE, BEADED
EQUIPPED WITH BLACKOUT CONTROL
4" WHITE BEADED-100' LONG
YELLOW GLASS BEAD BARRIER LINE 100' LONG
10'
4" WHITE BEADED-100' LONG
SILVER R C TYPE A 6" x 24" AT BASE
BLACK AND WHITE 12" STRIPES 5' UP FROM BASE OF POLE
4" WHITE LINES, BEADED, 3' LONG 2' C TO C.
EDGE LINE 100'
YELLOW BARRIER
EDGE LINE 100'
DRIVE TO FILLING STATION
BLACK AND WHITE CURB MARKINGS, 12" INTERVALS. BEADED PAINT USED IN WHITE INTERVALS
8" WHITE LINE, BEADED
C
SIDEWALK
BASIN STREET
CROSSWALK LINES
4'
BLACK AND WHITE BEADED CURB RETURNS
12" PLAIN WHITE LINES
CURB PAINTED WHITE, BEADED
D
CURB PAINTED BLACK AND WHITE 12" INTERVALS. WHITE INTERVALS BEADED OPPOSITE STREET ENTRANCE
SILVER R C TYPE A CHECKERED GUARD-RAIL BOARD
20' STALLS, 7' WITH TRANSVERSE LINES
4" WHITE CENTERLINE, BEADED
4" PLAIN WHITE CENTERLINE
CURB PAINTED BLACK AND WHITE, 12" INTERVALS
4" PLAIN WHITE LINE
7'
8" CROSSWALK LINE
4'
8" WHITE LINE, BEADED
4" WHITE LINES, BEADED
3' OR LESS, EVENLY DIVIDED
WATER STREET
10'
STOP
STOP SIGN
5' WORD STOP ON PAVEMENT
E
7' LINES PERPENDICULAR TO CURB LINE 20' APART. 4" WHITE, BEADED
6" CURB LINE PLAIN WHITE
FIREPLUG
CURB RETURN PAINTED WITH BLACK AND WHITE REFLECTORIZED VERTICAL STRIPES 12" WIDE
4" PLAIN WHITE LINES 20' APART. 7' LINES PERPENDICULAR TO CURB LINES
STREET
CHURCH
8" WHITE LINES, BEADED
CURBS PAINTED WHITE
STOP SIGN
FIREPLUG
20 STALLS-NO LINES ACROSS DRIVEWAYS
4" WHITE LINES, BEADED, 5' LONG, 2' C.TO C
25'
4" WHITE LINE, BEADED
4" PLAIN WHITE LINE
STALLS SAME AS OPPOSITE SIDE
WIDE PAVEMENT ENDS
EAST CORPORATE LIMIT OF HEBRON
F

FIGURE 3.—PAVEMENT AND SPECIAL MARKINGS ON URBAN SECTION OF TEST ROUTE.



INVESTIGATION OF HIGHWAY MARKINGS AND SIGNS FOR BLACKOUT CONDITIONS

QUESTIONNAIRE FOR PEDESTRIAN OBSERVERS
(To be filled out by Auxiliary Policemen, Air Raid Wardens, etc.)

A. Effectiveness of Blackout:

1. Were all interfering lights extinguished at time of test? Yes ✓ No ____
Location of exceptions ____

2. How many pedestrians did you see that were not participating in test? 3
Did they know about blackout? 2 did
If not, did you caution them? Yes. Did they cooperate? Yes

3. Were any unauthorized vehicles on the road? Yes ✓ No ____ How many? 1
Did they cooperate when informed? Yes—took side road

B. Visibility of Blackout Cars:

4. Was there any difficulty in detecting the approach of a blackout car? No

5. What was your first warning of an approaching car?
(a) Headlight ✓ (c) Horn ____
(b) Marker Lights ____ (d) Motor Noise ____

6. Did you recognize the marker lights? Yes ✓ No ____ Distance 100 Yards

7. Are vehicles with blackout equipment sufficiently visible to permit necessary pedestrians to move safely? Yes ✓ No ____

C. Movement of Vehicles Equipped with Blackout Equipment:

8. Did most drivers observe stop signs and signals? Yes ✓ No ____
Location and nature of violations None

9. Did cars appear to be driving dangerously fast? No

10. Did you notice any vehicles on wrong side of road? 2 at curve

D. Additional Comments:

Name of Observer O. A. Burton
Where stationed during test Post #10
Assigned duties during test Police
Date 8-8-42

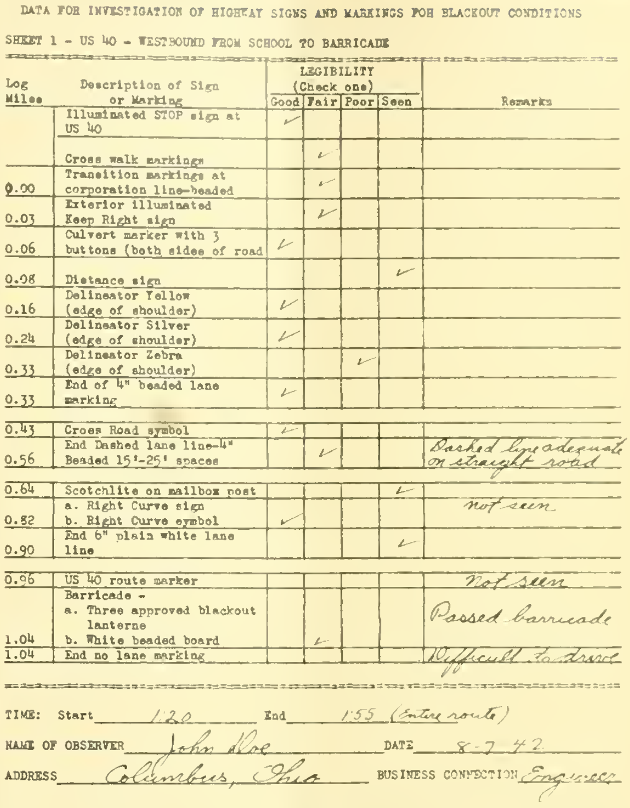
FIGURE 4.—QUESTIONNAIRE FORM FILLED OUT BY PEDESTRIAN OBSERVERS.

No data were recorded the first night except those obtained by having the emergency police and wardens fill out the pedestrian questionnaire (fig. 4). This preliminary test was valuable as a guide in determining the procedure to be followed during the succeeding tests and revealed a number of desirable changes in the markings and equipment, particularly in the adjustment of the blackout head lamps.

EXISTING SIGNS FOUND INADEQUATE

All head lamps were mounted with the horizontal slot in the mask 42 inches above the road surface and the center of the slot 11 inches to the left of the centerline of the vehicle, and were adjusted so that the top of the visual cut-off of the beam at a distance 10 feet ahead of the vehicle was 2 inches lower than the bottom of the horizontal slot in the head-lamp mask. With this adjustment, most of the pedestrian observers and drivers complained of being temporarily blinded by approaching test cars. Therefore, for subsequent tests the head lamps were adjusted so that the visual cut-off 10 feet ahead of the vehicle was between 2½ and 3 inches below the horizontal slot in the mask for a car with driver and one passenger in the front seat, and at least 2 inches below when there were also three people in the back seat. However, even with this adjustment, which was used during the following series of tests, it was not uncommon for a light beam to shine above the horizontal as the cars traveled along the relatively smooth surfaces of the test course, but this was a great improvement over the adjustment used during the preliminary test. Any further increase in the slope of the beam would have seriously affected the illumination provided by the head lamp.

The test runs conducted to record the driver's observations and reactions to the various signs and markings were made in the early morning hours on August 5 and 7. Fifteen trips over the test course were made on August 5 and eight trips on August 7. During the last seven trips on August 5, there was a quarter moon so the data obtained during these trips have been seggregated in the analyses. No brightness measurements were made on August 5 but those made by a representative of the General Electric Laboratories at 3 a. m. August 7 were as follows and indicate that the night was about as dark as any with starlight:



DATA FOR INVESTIGATION OF HIGHWAY SIGNS AND MARKINGS FOR BLACKOUT CONDITIONS

SHEET 1 - US 40 - WESTBOUND FROM SCHOOL TO BARRICADE

Log Miles	Description of Sign or Marking	LEGIBILITY (Check one)				Remarks
		Good	Fair	Poor	Seen	
	Illuminated STOP sign at US 40	✓				
	Cross walk markings		✓			
0.00	Transition markings at corporation line-beaded		✓			
0.03	Exterior illuminated Keep Right sign		✓			
0.06	Culvert marker with 3 buttons (both sides of road	✓				
0.08	Distance sign				✓	
0.16	Delineator Yellow (edge of shoulder)	✓				
0.24	Delineator Silver (edge of shoulder)	✓				
0.33	Delineator Zebra (edge of shoulder)			✓		
0.33	End of 4" beaded lane marking	✓				
0.43	Cross Road symbol	✓				
0.56	End Dashed lane line-4" Beaded 15'-25' spaces		✓			Dashed line adequate on straight road
0.64	Scotchlite on mailbox post				✓	
	a. Right Curve sign					not seen
0.82	b. Right Curve symbol	✓				
0.90	End 6" plain white lane line				✓	
0.96	US 40 route marker					Not seen
1.04	Barricade - a. Three approved blackout lanterns b. White beaded board		✓			Passed barricade
1.04	End no lane marking					Difficult to drive

TIME: Start 1:20 End 1:55 (Entire route)

NAME OF OBSERVER John Doe DATE 8-7-42

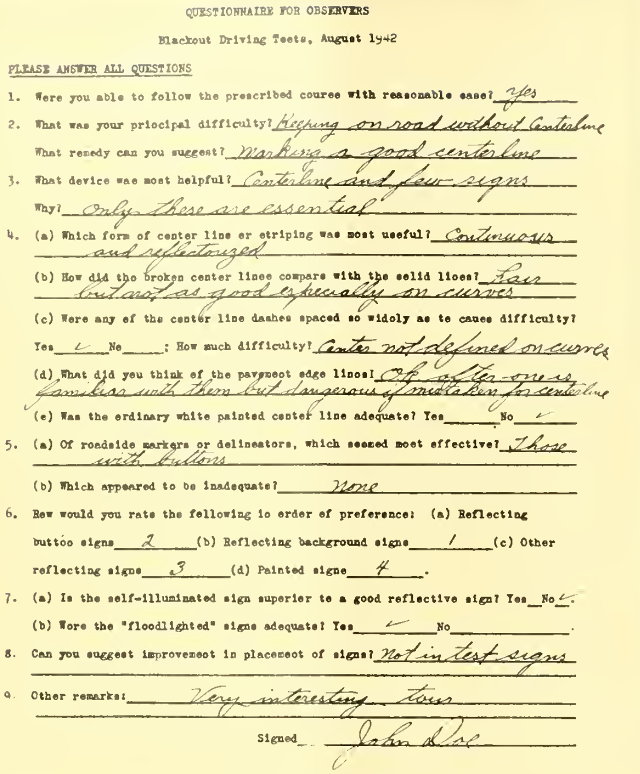
ADDRESS Columbus, Ohio BUSINESS CONNECTION Engineer

FIGURE 5.—FIRST SHEET OF 16-PAGE LOG USED TO RECORD DRIVER REACTION TO SPECIAL SIGNS AND MARKINGS.

Object:	Footlamberts
Zenith sky	0. 000076
Sky-45° to west	. 000027
Concrete surface	. 000010

The procedure followed was to assign a driver, guide, recorder, and one or two observers to each test car. The drivers and observers consisted of men from a large variety of occupations, none familiar with the test route. The guides and recorders were familiar with the test route. The guide sat alongside the driver and was present mainly for the purpose of showing the driver the course of the route, to warn the driver of pedestrians or dangerous locations if the speed was excessive and in general to see that the trip was made safely. The recorder, who rode in the back seat and was equipped with a very dim flashlight, entered the remarks made by the driver on a 16-page log (fig. 5) listing each traffic device, marking, or sign on the route. There were 260 listed items including such items as center-line markings that were listed twice. The drivers were not expected to rate or remark about all items but if they did, space was available on the form. No remarks made by the observers during the trip were recorded but both the drivers and observers filled out the form shown as figure 6 when the trip was completed.

Prior to the start of the trip, the purpose of the tests and the procedure to be followed were explained to the drivers and observers. The drivers were requested to maintain a speed between 15 and 20 miles per hour and



QUESTIONNAIRE FOR OBSERVERS

Blackout Driving Tests, August 1942

PLEASE ANSWER ALL QUESTIONS

1. Were you able to follow the prescribed course with reasonable ease? Yes

2. What was your principal difficulty? Keeping on road without Centerline

What remedy can you suggest? Marking a good centerline

3. What device was most helpful? Centerline and few signs

Why? Only these are essential

4. (a) Which form of center line or striping was most useful? Continuous and reflectorized

(b) How did the broken center lines compare with the solid lines? Fair but not as good especially on curves

(c) Were any of the center line dashes spaced so widely as to cause difficulty?

Yes ✓ No ____ ; How much difficulty? Center not defined on curves

(d) What did you think of the pavement edge lines? Ok after one is familiar with them but dangerous if mistaken for centerline

(e) Was the ordinary white painted center line adequate? Yes ____ No ✓

5. (a) Of roadside markers or delineators, which seemed most effective? Those with buttons

(b) Which appeared to be inadequate? None

6. How would you rate the following in order of preference: (a) Reflecting button signs 2 (b) Reflecting background signs 1 (c) Other reflecting signs 3 (d) Painted signs 4 .

7. (a) Is the self-illuminated sign superior to a good reflective sign? Yes ____ No ✓.

(b) Were the "floodlighted" signs adequate? Yes ✓ No ____.

8. Can you suggest improvement in placement of signs? Not in test signs

9. Other remarks: Very interesting tour

Signed John Doe

Figure 6.—Questionnaire Form Filled Out by Drivers and Observers.

to remark about each traffic line, sign, or other device that they saw, and when possible to give a rating such as "good," "fair," or "poor," based on the visibility and effectiveness of the particular device. The eyes of everyone were "dark adapted" prior to starting over the test course. In addition, a night vision test was given most of the drivers and observers. This involved determining what line on an optometrist's chart, faintly illuminated, could be read by the subject at a distance of 10 feet in a blacked-out room. While this was a very rough test, it was desired to obtain some measure of the relative night vision of the various drivers.

Table 1 [5] shows the results of the observations made on standard signs placed, as to distance from the centerline and along the roadway, in the manner used for signing highways during normal conditions. The signs were along 2-lane highways with the exception of the railroad sign, which was on a 4-lane road, and the 3 signs placed 18 feet from the centerline which were on a 3-lane road. Most of the signs included were mounted at standard heights but a few were placed for this test as near the pavement elevation as possible.

In addition to the ratings of "good," "fair," and "poor" that were applied to signs that were legible, the rating "seen" was used when the observer saw the sign but could not read the legend. The rating "not seen" was applied when the driver made no mention of the sign as he passed it.

Rating factors are shown for each type of sign and placement classification in order to have a single figure for comparative purposes. They have been calculated on the basis that a sign rated as "good" by all drivers should have a rating factor of 100 and one not seen by any of the drivers should have a rating factor of 0. Since it was felt that a sign that could not be read, but that could be seen, has some value, some weight has been given to the "seen" ratings. Other methods of calculating a rating factor were also tried but for all those that seemed logical, approximately the same results were obtained.

SIGNS MUST BE PROPERLY REFLECTORIZED

Considering first the plain painted signs mounted at standard heights, table 1 shows that the route markers were not seen 72 percent of the time, the information signs were not seen 61 percent of the time, and the symbol signs were not seen 39 percent of the time. The majority of observers that did see signs in these three groups either could not distinguish the legends or rated the legibility as "poor." This resulted in rating factors of 10, 13, and 25 for the three groups of plain painted signs.

Reflectorizing the signs mounted at a standard height with either glass reflecting buttons or a reflectorized material improved their legibility very little as may be seen by the results for the "symbol," "WIDE PAVEMENT ENDS," and "railroad crossing" signs. However, the data for the few signs that were both reflectorized and mounted with their top edges within 2 feet of the elevation of the road surface indicate that this treatment resulted in a marked increase in the legibility of the standard signs. For example, the rating factor for the route markers increased from 10 to 38 (still too low for a sign to be effective) while the rating factor for the 24-inch "stop" sign was 81, a very high rating considering that the sign was on a 3-lane road and to receive a "good" rating the operator had to bring the vehicle to a stop before reaching a limit line directly opposite the sign.

Although the legibility ratings on table 1 and those on all succeeding tables regarding signs are generally lower for the moonlight condition (whenever there is an appreciable difference) than for the condition without moonlight, the results are not consistent. The moonlight on August 5 apparently had little or no effect on the legibility of signs as compared to the effect of other factors.

Thirty of the 42 signs made especially for this test by the Ohio Department of Highways were mounted along the shoulder on tangent sections of a 20-foot concrete road. All were placed 16 feet from the center line, at an angle of 10° toward the road from a line perpendicular to the centerline, and at a height so that the bottom edge of the sign was approximately at the same elevation as the road crown (fig. 7–A). All of these signs were 2 feet square and had legends consisting of four-letter words 6 inches high. By using different types and colors of materials and reflectorizing the letters on some and the background on others, 19 different combinations were used. Eleven signs were each duplicates of 1 of the other 19 except for the legend. In placing the signs, care was taken so that no sign could be seen from a vehicle on a curve or close enough to another sign to be visible while the other sign was still in view.

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[5] As many materials as were available at the Ohio Highway Department's paint shop or could be obtained within the time permitted for this study were used to construct the signs. Table 1 shows that three types of reflectorized coatings (R. C.) were used. Types A and B designate the products of two different manufacturers and include reflectorized flexible materials that are applied to the face of the sign, while type C includes the coating obtained by the Ohio State Highway Department's sign shop by sprinkling glass beads over the surface of a freshly painted sign. The same designations apply when used in connection with other tables and the text of this report. Before or after these studies were made, products better suited for particular conditions may have been placed on the market by the same or other manufacturers.

Table 1.—*Legibility of standard signs*

Type of sign	Placement		Color and material [2]		Distribution of legibility ratings							
	Mounting height [1]	Distance from center of road	Letters	Background	Sky condition	Number of observations	Good	Fair	Poor	Seen	Not seen	Rating factor [3]
	Feet	*Feet*					*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	
Route markers	5	16	Black	White	Dark	150	2	3	11	13	71	11
					Moon	60	0	3	18	3	76	8
					Total	210	1	3	13	11	72	10
Do	2	16	do	do	Dark	45	4	7	27	16	46	21
					Moon	18	11	17	11	6	55	25
					Total	63	6	10	22	13	49	22
Do	2	16	do	Transparent R. C. type A.	Dark	45	16	22	20	29	13	41
					Moon	18	11	6	39	17	27	31
					Total	63	14	18	25	25	18	38
Information	4.5	16	do	White	Dark	15	2	2	27	11	58	15
					Moon	21	0	0	24	9	67	10
					Total	66	1	1	26	11	61	13
Symbol	4.5	16	do	Yellow	Dark	112	9	5	36	13	37	27
					Moon	56	5	4	27	20	44	21
					Total	168	8	5	33	15	39	25
Do	4.5	16	do	Yellow R. C. type B	Dark	15	33	7	13	20	27	46
					Moon	6	0	17	33	0	50	20
					Total	21	24	10	19	14	33	39
Do	4.5	16	Black with glass reflecting buttons.	Yellow	Dark	82	6	16	30	10	38	27
					Moon	42	3	9	26	14	48	20
					Total	124	5	14	29	11	41	25
Wide pavement ends	4.5	18	Black	Yellow R. C. type C	Dark	15	0	0	33	13	54	11
					Moon	6	0	0	11	0	86	5
					Total	21	0	0	27	9	64	11
Railroad crossing	4.5	28	Black with glass reflecting buttons.	Yellow	Dark	15	20	0	33	0	17	31
					Moon	6	0	28	0	11	58	18
					Total	21	14	9	23	5	49	28
Distance	2	16	Black	Silver R. C. type A	Dark	15	33	40	20	0	7	60
					Moon	6	33	33	17	17	0	60
					Total	21	33	38	19	5	5	60
30-inch stop	2.5	18	Silver R. C. type A with black border.	Yellow	Dark	15	27	13	14	13	35	42
					Moon	6	0	0	0	33	67	8
					Total	21	19	10	10	19	42	32
Keep right	2	0	Black with glass reflecting buttons.	White	Dark	8	63	13	0	12	12	73
24-inch stop	2	18	White with plastic reflecting buttons on black panel.	Yellow	Dark	15	80	7	0	0	13	84
					Moon	6	50	33	17	0	0	72
					Total	21	72	14	5	0	9	81

[1] To top of sign from crown of road.

[2] R. C. represents material with a reflectorized coating.

[3] Obtained by adding all of the percentage rated "good," one-half the percentage rated "fair," one-third the percentage rated "poor," and one-fourth the percentage rated "seen."

Table 2 shows the distribution of legibility ratings and rating factors for the special test signs. The legibility ratings of the plain painted signs (group 1) were all too low for any of these signs to be of material aid to a driver under blackout conditions.

Of the signs with plain black letters and a background made of material with a reflectorized coating (group 2, table 2), those with white material rated consistently higher than those with yellow material made by the same manufacturer although there was not a large difference. For both the white and yellow material, type A, type C, and type B rated first, second, and third, respectively, although the differences were slight. However, the most important finding was that the rating factor for the sign with the silver reflectorized coating (type A) was almost perfect and nearly twice as high as the average rating for any of the other materials.

Signs having letters constructed of a reflectorized material (group 3, table 2) and mounted on a black panel 8 inches high attached to a plain painted background all received comparatively high legibility ratings. It should be noted, however, that only the two reflectorized materials receiving the highest ratings in group 2 were used for the signs in group 3. Here again, the silver-colored material (type A) received higher ratings than the white material when both were on the same backgrounds. The signs with the white backgrounds rated higher than those with the yellow backgrounds when the letters were of the same material. The only explanation for this is that a white background provided a greater contrast with the black panel and increased the target value of the sign, thus helping to draw the driver's attention to the sign before the letters were fully illuminated.

None of the signs with reflector buttons received a

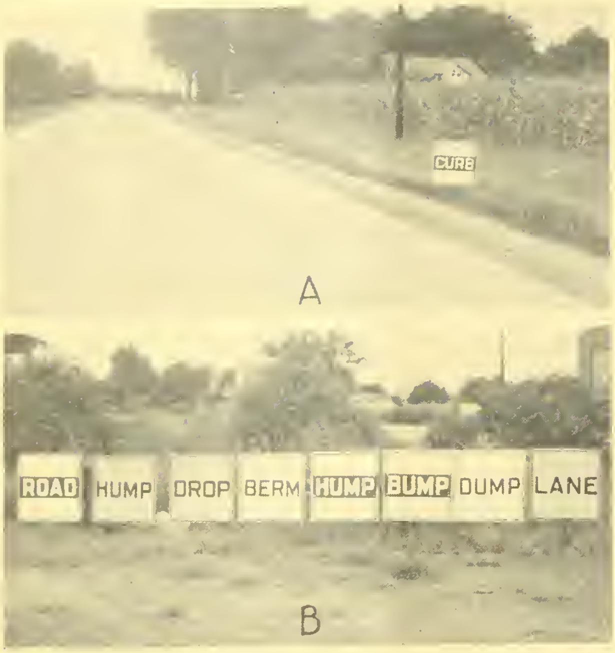

FIGURE 7.—SPECIAL SIGNS USED FOR TESTS; A, PLACED ALONG ROUTE; B, BATTERY OF SIGNS VIEWED FROM STATIONARY VEHICLE.

TABLE 2.—*Legibility of test signs, placed at an angle of 10° to the road, low, and 16 feet from the centerline*

GROUP 1—PLAIN PAINTED SIGNS

Color and material[1]		Sky condition	Number of observations	Distribution of legibility ratings					Rating factor[2]
Letters	Background			Good	Fair	Poor	Seen	Not seen	
				Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	
Black	White	Dark	30	0	13	33	37	17	27
		Moon	12	0	8	50	34	8	29
		Total	42	0	12	38	36	14	28
Do	Yellow	Dark	37	3	13	41	35	8	32
		Moon	18	5	17	39	28	11	34
		Total	55	4	14	40	33	9	33
White on black panel.	White	Dark	15	7	20	47	26	0	39
		Moon	6	17	17	33	0	33	37
		Total	21	10	19	42	19	10	38
Do	Yellow	Dark	15	7	7	53	33	0	37
		Moon	6	0	17	67	0	16	31
		Total	21	5	9	57	24	5	35
	Combined	Dark	97	4	13	44	33	6	34
		Moon	42	6	15	47	16	16	33
		Total	139	5	14	44	28	9	34

GROUP 2—BACKGROUND OF MATERIAL WITH REFLECTORIZED COATING

Letters	Background	Sky condition	Number of observations	Good	Fair	Poor	Seen	Not seen	Rating factor
Black	Silver R. C. type A.	Dark	30	93	7	0	0	0	97
		Moon	12	92	8	0	0	0	96
		Total	42	93	7	0	0	0	97
Do	White R. C. type A.	Dark	15	33	33	27	0	7	59
		Moon	6	17	33	50	0	0	50
		Total	21	29	33	33	0	5	57
Do	Yellow R. C. type A.	Dark	15	33	27	27	0	13	56
		Moon	6	0	67	33	0	0	45
		Total	21	24	38	29	0	9	53

TABLE 2.—*Legibility of test signs, placed at an angle of 10° to the road, low, and 16 feet from the centerline*—Continued

GROUP 2—BACKGROUND OF MATERIAL WITH REFLECTORIZED COATING—Continued

Color and material		Sky condition	Number of observations	Distribution of legibility ratings					Rating factor
Letters	Background			Good	Fair	Poor	Seen	Not seen	
			Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Black	White R. C. type C.	Dark	30	27	36	10	20	7	54
		Moon	12	42	42	0	16	0	67
		Total	42	31	38	7	19	5	57
Do	Yellow R. C. type C.	Dark	30	24	30	20	13	13	49
		Moon	12	17	42	25	16	0	50
		Total	42	22	33	21	14	10	50
Do	White R. C. type B.	Dark	22	27	0	64	9	0	51
		Moon	12	17	33	42	0	8	48
		Total	34	23	12	56	6	3	49
Do	Yellow R. C. type B.	Dark	15	13	40	40	7	0	48
		Moon	6	0	67	33	0	0	45
		Total	21	9	48	38	5	0	47
	Combined	Dark	157	35	25	27	7	6	58
		Moon	66	26	42	26	5	1	57
		Total	223	33	30	26	6	5	58

GROUP 3—LETTERS OF MATERIAL WITH REFLECTORIZED COATING

Letters	Background	Sky condition	Number of observations	Good	Fair	Poor	Seen	Not seen	Rating factor
Silver R. C. type A.	Black panel on white.	Dark	15	86	7	0	7	0	91
		Moon	6	100	0	0	0	0	100
		Total	21	90	5	0	5	0	94
Do	Black panel on yellow.	Dark	7	57	29	0	0	14	72
		Moon	6	67	17	0	16	0	80
		Total	13	62	23	0	8	7	76
White R. C. type A.	Black panel on white.	Dark	30	63	27	7	3	0	80
		Moon	12	59	33	8	0	0	78
		Total	42	62	29	7	2	0	79
Do.	Black panel on yellow.	Dark	30	37	50	3	3	7	64
		Moon	12	25	42	25	0	8	54
		Total	42	33	48	10	2	7	61
	Combined	Dark	82	61	28	3	3	5	77
		Moon	36	63	23	8	4	2	79
		Total	118	62	26	4	4	4	77

GROUP 4—LETTERS WITH GLASS REFLECTING BUTTONS

Letters	Background	Sky condition	Number of observations	Good	Fair	Poor	Seen	Not seen	Rating factor
Black with buttons.	White	Dark	30	43	47	3	7	0	70
		Moon	12	8	59	25	0	8	46
		Total	42	33	50	10	5	2	63
Do	Yellow	Dark	15	20	40	13	7	20	46
		Moon	6	16	50	17	0	17	47
		Total	21	19	43	14	5	19	47
White with buttons.	Black panel on white.	Dark	15	53	47	0	0	0	77
		Moon	6	33	67	0	0	0	67
		Total	21	48	52	0	0	0	74
Do	Black panel on yellow.	Dark	30	67	33	0	0	0	84
		Moon	12	42	50	8	0	0	70
		Total	42	60	38	2	0	0	80
	Combined	Dark	90	46	42	4	3	5	69
		Moon	36	25	57	12	0	6	58
		Total	126	40	46	7	2	5	66

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[1] "R. C." represents material with a reflective coating.

[2] Obtained by adding all of the percentage rated "good", one-half the percentage rated "fair", one-third the percentage rated "poor", and one-fourth the percentage rated "seen".

high a rating factor as the best sign in either group 2 or group 3 and the yellow sign with black reflectorized letters had as low a rating as any of the reflectorized signs. One probable reason why the signs with reflector buttons as a group did not receive better legibility ratings was that they were all located in succession along the test route. The ratings, which were necessarily relative values, were made in comparison with other signs of the same group, and were therefore not as high as they might have been had they been placed in company with a number of less legible signs.

At one of the turn-around areas, eight test signs were mounted side by side just above ground level (fig. 7–B). As each test vehicle turned, the driver stopped with the car facing the battery of signs, and as far away as it was possible for him to read the legends on most of the signs. He then rated the signs in the order of preference.

The results for the test on the battery of signs are shown in table 3. The preference rating factors have been calculated so that if a particular sign had been preferred by all observers its rating factor would have been 8, and had it been considered the poorest sign by all observers it would have had a rating factor of 1. There was little difference in the factors for the three outstanding signs. They were: The sign with black letters on a white reflectorized background, the sign with silver reflectorized letters on a black panel, and the sign with white letters reflectorized with buttons on a black panel. The sign shown as receiving the highest rating in table 2 (black letters on a silver reflectorized background) was not included in this test. Signs with yellow backgrounds, either reflectorized or painted, did not receive as high rating factors as did those with white backgrounds.

The results of these tests definitely prove that the legibility of reflectorized signs when viewed either from a moving or stationary vehicle varies with the type of reflecting material used, the color combinations used in the sign, and the location of the sign with respect to the roadway.

Three phosphorescent signs, previously energized by 12 hours of daylight and sunlight, and mounted above the battery of signs were not noticed by the observers. Three reflectorized signs mounted at the standard height of 3½ feet above the road (to center of sign) but near the pavement edge were either not noticed or rated as "poor" by the observers.

MOUNTING AND PLACEMENT OF SIGNS IMPORTANT

Two types of illuminated signs meeting War Department Specifications for blackout conditions[6] were tested. One type (fig. 8–A) was an interior-illuminated sign in the shape of a box 19 inches long, 6 inches wide, and 4½ inches high. The transparent face of the sign was painted black with the exception of the legend which was of rounded letters 3 inches high made with a stroke ½ inch wide. Inside the box were a No. 6 dry cell battery and three 1.35-volt 0.06 ampere bulbs that will operate continuously for about 1 week with either a No. 6 dry cell or 10 standard flashlight batteries in parallel. Both ground glass and opal glass were used for the face of these signs. Three signs of each material were mounted along the test route at a height of 4½ to 6 feet above the pavement surface.

The other type of illuminated sign was provided by mounting a light 15 inches in front and slightly below

TABLE 3.—*Relative legibility of test signs mounted low in front of stationary vehicle*

Color and material		Sky conditions	Number of observations	Preference								Preference rating factor[2]
Letters[1]	Background			1st	2d	3d	4th	5th	6th	7th	8th	
				Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	
Black	White	Dark	15	0	7	0	7	0	13	73	0	2.7
		Moon	6	0	0	17	0	0	17	66	0	2.9
		Total	21	0	5	5	5	0	14	71	0	2.7
Do.	Yellow	Dark	15	0	0	0	7	0	0	33	60	1.6
		Moon	6	0	0	17	0	17	32	17	17	3.2
		Total	21	0	0	4	4	5	10	29	48	2.0
White	Black panel on white	Dark	15	0	0	0	0	13	53	27	7	2.7
		Moon	6	0	0	0	17	17	17	17	32	2.7
		Total	21	0	0	0	5	14	43	24	14	2.7
Black	White R. C. type A	Dark	15	27	46	7	7	13	0	0	0	6.7
		Moon	6	33	17	33	0	17	0	0	0	6.5
		Total	21	29	38	14	5	14	0	0	0	6.6
Do.	Yellow R. C. type A	Dark	15	0	7	33	40	20	0	0	0	5.3
		Moon	6	0	33	50	17	0	0	0	0	6.2
		Total	21	0	14	39	33	14	0	0	0	5.5
Do	Yellow R. C. type B	Dark	15	7	0	7	27	45	7	0	7	4.4
		Moon	6	0	0	0	17	17	17	17	32	2.7
		Total	21	5	0	5	21	37	10	[illegible]	14	[illegible]
Silver R. C. type A	Black panel on white	Dark	15	20	53	13	0	7	0	7	[illegible]	[illegible]
		Moon	6	32	0	17	17	17	0	17	[illegible]	[illegible]
		Total	21	24	37	14	5	10	[illegible]	10	[illegible]	[illegible]
White buttons	do	Dark	15	40	13	27	13	[illegible]	7	0	[illegible]	[illegible]
		Moon	6	17	33	[illegible]	17	[illegible]	[illegible]	0	[illegible]	[illegible]
		Total	21	33	[illegible]	29	14	0	[illegible]	0	[illegible]	[illegible]

---

[6] Traffic Control During Blackouts, prepared under the direction of the Chief of Engineers, United States Army, by the Engineer Board.

[1] R. C. = reflectorized coating.

[2] Calculated by obtaining a summation of 8 times the percentage of the first preference, 7 times the [illegible] second [illegible]

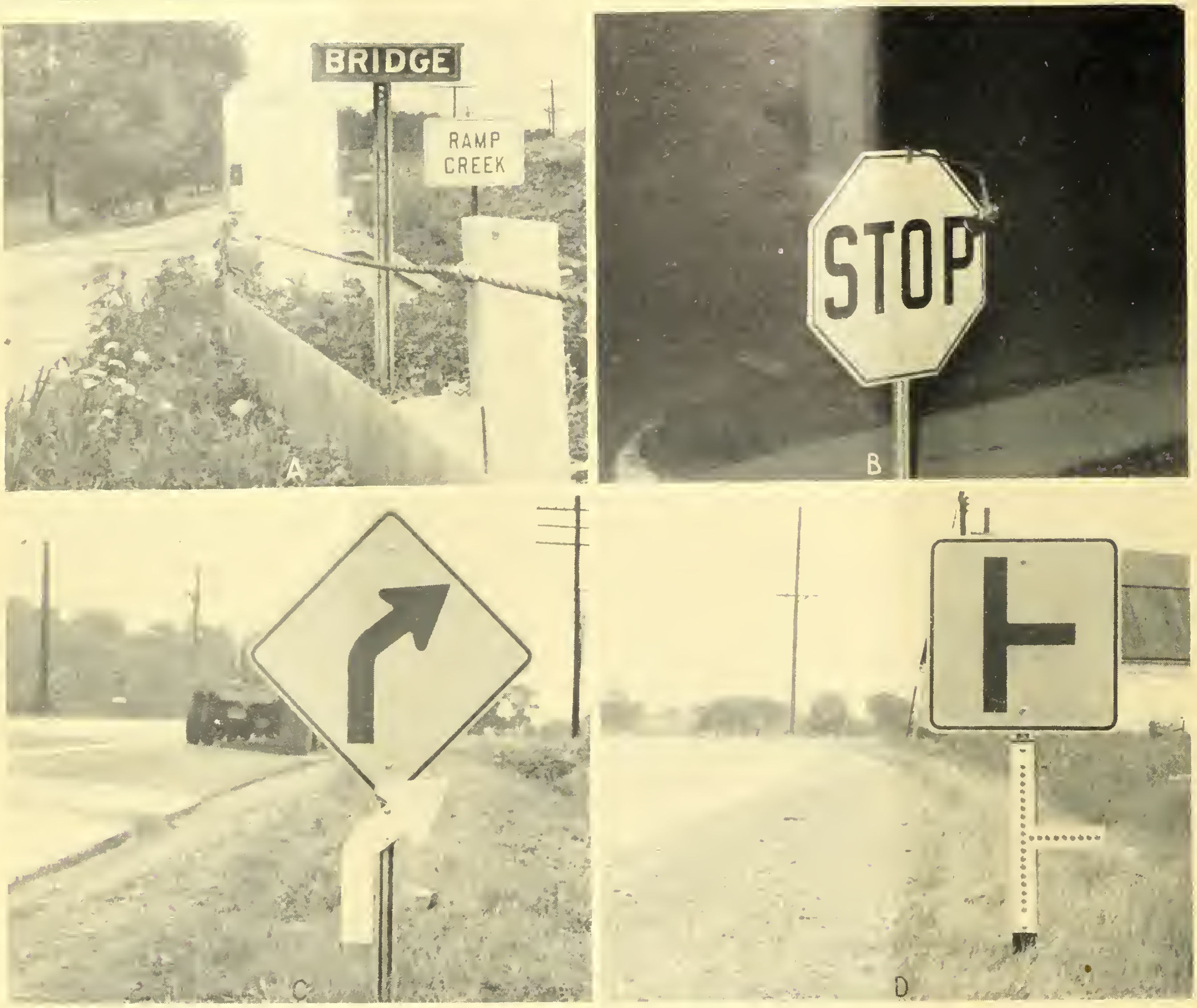


Figure 8.—Traffic Signs Tested Under Blackout Conditions. A, Interior-Illuminated Bridge Sign; B, Standard Stop Sign Illuminated by Exterior Light; C, Arrow Symbol With Reflectorized Coating Mounted Below Standard Sign; D, Side Road Symbol Reflectorized With Glass Buttons Mounted Below Standard Sign.

the top of a standard nonreflectorized sign so that the legend was illuminated. The light consisted of a 1.35-volt, 0.06-ampere bulb with a flashlight reflector painted white. Two types of mountings were used. One consisted of a single-battery flashlight, with the lens removed, held in position by an arm extending out from the signpost (fig. 8–B). The other consisted of a curved metal arm with a clamp holding a No. 6 dry cell and light bulb that fastened to the top of the sign. Both the battery and light bulb were grounded on the metal arm thus requiring only one insulated wire to complete the electrical circuit.

Table 4 shows the results of tests with both types of illuminated signs. The interior-illuminated signs with opal glass were far superior to those with ground glass. They were also superior to the exterior-illuminated signs. Of the exterior-illuminated signs, the ones with black letters on a yellow background were better than those with black letters on a white background.

Table 5 shows the results obtained for a variety of special symbols mounted within 24 inches of the elevation of the pavement surface (figs. 8–C and 8–D). These symbols had no background and were made by cutting the particular shape desired from a piece of weatherproof composition board such as masonite and then painting or reflectorizing one surface. They were generally mounted farther from the pavement surface than the special test signs and were located on existing mail-box posts, telephone poles, or below standard signs along the test route. The rating factors for the symbols were much higher than for the standard signs and those for some of the symbols compare favorably with the ratings for the special test signs considering that the symbols were sometimes mounted at a greater lateral distance from the highway surface. Direct comparisons cannot be made for the different types of reflecting materials since there was such a large variation in the mounting locations.

Prior to the installation of the special signs on the test route, tests were made at the General Electric Co. laboratory at Nela Park to determine the distance from a stationary vehicle that various signs were legible and the effect of mounting height, angularity, and transverse position upon legibility. The test signs

TABLE 4.—*Legibility of interior- and exterior-illuminated signs*

INTERIOR-ILLUMINATED SIGNS

Material used	Sky condition	Number of observations	Distribution of legibility ratings					Rating factor [1]
			Good	Fair	Poor	Seen	Not seen	
			Percent	*Percent*	*Percent*	*Percent*	*Percent*	
Ground glass	Dark	45	35	27	22	7	9	58
	Moon	18	28	22	22	6	22	48
	Total	63	33	26	22	6	13	55
Opal glass	Dark	45	60	13	9	7	11	71
	Moon	18	55	39	0	6	0	76
	Total	63	59	21	6	6	8	73

EXTERIOR-ILLUMINATED PAINTED SIGNS

Material used	Sky condition	Number of observations	Good	Fair	Poor	Seen	Not seen	Rating factor [1]
Black on white	Dark	67	13	33	30	6	18	41
	Moon	30	27	33	14	3	23	49
	Total	97	17	33	25	5	20	43
Black on yellow	Dark	23	43	22	9	9	17	59
	Moon	6	17	67	0	16	0	55
	Total	29	38	31	7	10	14	58

[1] Obtained by adding all of the percentage rated "Good," one-half the percentage rated "Fair," one-third the percentage rated "Poor," and one-fourth the percentage rated "Seen."

TABLE 5.—*Visibility of special symbols mounted low*

CROSSROAD, SIDE ROAD, AND CURVE SYMBOLS

Color and material used	Sky condition	Number of observations	Distribution of visibility ratings					Rating factor [1]
			Good	Fair	Poor	Seen	Not seen	
			Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	
Plain white paint	Dark	60	22	10	23	10	35	37
	Moon	24	21	12	21	0	46	34
	Total	84	22	11	22	7	38	37
Silver R. C. type A	Dark	45	60	11	4	0	25	67
	Moon	18	33	17	0	17	33	46
	Total	63	52	13	3	5	27	61
Glass reflecting buttons on white paint	Dark	60	57	15	8	12	8	70
	Moon	24	63	8	4	4	21	69
	Total	84	58	13	7	10	12	69
Glass reflecting buttons on yellow R. C. type A.	Dark	45	33	5	13	7	42	42
	Moon	18	17	17	11	5	50	30
	Total	63	29	9	12	6	44	40
Glass reflecting buttons on silver R. C. type A.	Dark	15	40	0	13	7	40	46
	Moon	6	0	67	0	0	33	34
	Total	21	29	19	9	5	38	43

NIGHT ARROW SYMBOLS

Color and material used	Sky condition	Number of observations	Good	Fair	Poor	Seen	Not seen	Rating factor [1]
Plain white paint	Dark	15	0	13	13	0	74	11
	Moon	6	0	33	33	0	34	28
	Total	21	0	19	19	0	62	16
Silver R. C. type A	Dark	15	27	0	0	13	60	30
	Moon	6	67	0	0	33	0	75
	Total	21	38	0	0	19	43	43
Glass reflecting buttons on white paint	Dark	15	27	6	20	20	27	42
	Moon	6	17	33	17	0	33	39
	Total	21	24	14	19	14	29	41
Glass reflecting buttons on yellow R. C, type A.	Dark	15	60	7	0	0	33	64
	Moon	6	50	0	0	17	33	54
	Total	21	57	5	0	5	33	61

[1] Obtained by adding all of the percentage rated "Good", one-half the percentage rated "Fair", one.third the percentage rated "Poor" and one-fourth the percentage rated "Seen".

were viewed from the driver's seat of one of the test vehicles with the blackout head lamp adjusted so that the visual cut-off of the top of the light beam sloped down ahead of the car at the rate of 2 inches every 10 feet from a height of 42 inches.

The results of these observations of signs viewed from a stationary vehicle were:

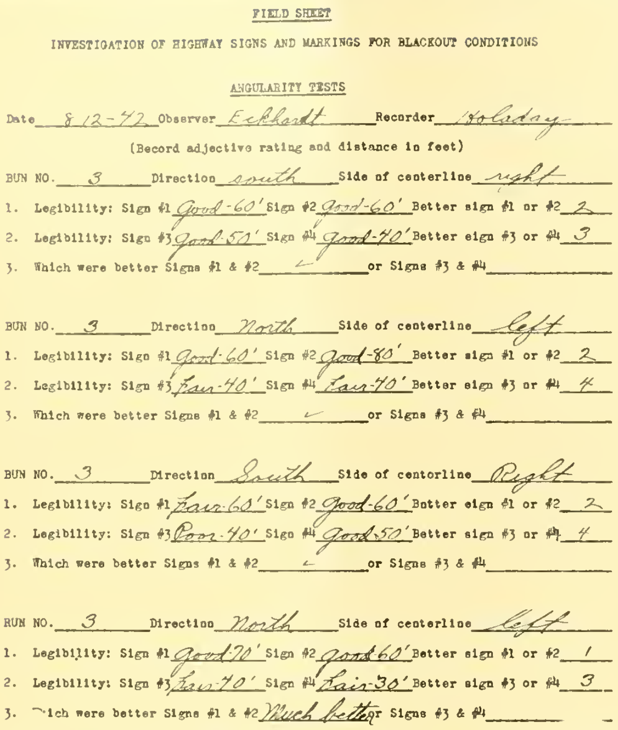
1. To be legible, a sign must be mounted within the range of the headlight beam.
2. For the 6-inch legend on a 24-inch square sign to be within the headlight beam, the maximum distance that the sign could be mounted ahead of the vehicle was 135 feet.
3. At a distance of 100 feet, the legend was illuminated when the bottom of the sign was 7 inches from the ground although better legibility was attained when the bottom of the sign was at ground level.
4. To be legible at 100 feet, a sign must be reflectorized. All reflectorized signs were not legible at 100 feet, however. (Some reflectorized signs with 6-inch letters are legible at distances exceeding 200 feet when the head-lamp beam is raised so that it strikes the sign.)
5. At a distance of 25 feet from the vehicle and 7 feet to the right of the centerline of the car, the 24-inch square reflectorized signs were legible when the bottom of the sign was placed anywhere from ground level to 26 inches above ground level, but the best legibility was obtained when 9 inches above ground level.
6. Sign legibility was not greatly affected by horizontal placement up to a distance of 18 feet to the right of the centerline of the vehicle (18 feet was the limit possible in the laboratory) when the sign was placed from 25 to 100 feet in front of the vehicle.
7. The best legibility was attained when the signs were turned so that they were normal to the line of sight of the observer.

The results of these tests were made use of when the special test signs were placed along the test route. However, since it is impossible to install signs so that they will always be normal to the driver's line of vision from a moving vehicle, and since the distance ahead of the vehicle that the signs were legible when placed at various distances from the edge of the road were not recorded, a separate study was made on August 12 to obtain information regarding:

1. The distance that signs suitable for use during blackouts are legible when viewed from a moving vehicle on both 2- and 4-lane highways.
2. The angle with both 2- and 4-lane roadways that signs should be mounted to obtain the best legibility from a moving vehicle.

INTERIOR-ILLUMINATED SIGNS FOUND MOST LEGIBLE

A level tangent section of two-lane highway with a smooth surface 20 feet wide was selected for this study. Normal traffic was detoured during the tests, and there were no lights in the neighborhood of the test section. A man with a number of test signs was stationed at each of eight locations (four on each side of the roadway) along the test section to change the sign at his particular location for each test run according to a prearranged schedule. The schedules were arranged so that direct comparisons could be made between similar signs placed at different angles with the roadway and between different signs placed at the same angle. None of the observers knew what sign was to be placed at a particular location.



FIELD SHEET

INVESTIGATION OF HIGHWAY SIGNS AND MARKINGS FOR BLACKOUT CONDITIONS

ANGULARITY TESTS

Date 8-12-42 Observer Eckhardt Recorder Holaday

(Record adjective rating and distance in feet)

RUN NO. 3 Direction south Side of centerline right
1. Legibility: Sign #1 Good-60' Sign #2 Good-60' Better sign #1 or #2 2
2. Legibility: Sign #3 Good-50' Sign #4 Good-40' Better sign #3 or #4 3
3. Which were better Signs #1 & #2 ✓ or Signs #3 & #4

RUN NO. 3 Direction North Side of centerline left
1. Legibility: Sign #1 Good-60' Sign #2 Good-80' Better sign #1 or #2 2
2. Legibility: Sign #3 Fair-40' Sign #4 Fair-40' Better sign #3 or #4 4
3. Which were better Signs #1 & #2 ✓ or Signs #3 & #4

RUN NO. 3 Direction South Side of centerline Right
1. Legibility: Sign #1 Fair-60' Sign #2 Good-60' Better sign #1 or #2 2
2. Legibility: Sign #3 Poor-40' Sign #4 Good-50' Better sign #3 or #4 4
3. Which were better Signs #1 & #2 ✓ or Signs #3 & #4

RUN NO. 3 Direction North Side of centerline left
1. Legibility: Sign #1 Good-70' Sign #2 Good-60' Better sign #1 or #2 1
2. Legibility: Sign #3 Fair-40' Sign #4 Fair-30' Better sign #3 or #4 3
3. Which were better Signs #1 & #2 Much better or Signs #3 & #4

Figure 9.—Form Used in Recording Legibility Distance and Adjective Ratings for Signs Placed at Different Angles.

All signs were mounted with their centers 2 feet outside the pavement edge. The mounting height from the elevation of the pavement crown to the center of the legend or symbol was 18 inches for the non-illuminated signs, 42 inches for the exterior-illuminated signs, and 54 inches for the interior-illuminated signs.

Figure 9 shows the form used to record the field data. The driver, as a sign was approached, called out the legend as soon as he could read it. The recorder immediately estimated the distance to the sign with the aid of small stakes covered with a reflecting material and located 30, 60, and 90 feet from the sign. While the driver may have actually been able to read the sign shortly before he called out the legend, his reaction time to do this was probably about the same as the time required for him to react in maneuvering the vehicle. In any case, the estimated distances were sufficiently accurate for this test.

After having passed the sign, the driver also gave the sign an adjective rating based on the distance that the sign remained legible and the relative legibility as compared with other signs. Although the signs that were legible from the greater distances generally received the higher adjective ratings, this did not always hold. For example, a sign that may have been legible for a long distance but was not outstanding at any distance may have been given a lower adjective rating than a sign that was very outstanding for a shorter distance. Data were obtained for both 2- and 4-lane highways by each observer driving over the test section twice for a particular sign installation, once to the right and once to the left of the centerline. Only the signs receiving the highest legibility ratings on the previous test route were included in this study.

Table 6 shows the legibility distances and ratings for the signs when at various angles with the roadway for 2- and 4-lane highways. The results, which were in line with but more conclusive than the previous sign tests, may be summarized as follows:

1. Interior-illuminated signs were the most legible. The average legibility distance was in excess of 100 feet whether the signs were 12 or 22 feet from the centerline of the roadway. The minimum legibility distance recorded was 50 feet, which is more than ample distance in which to stop a passenger car in good condition and traveling 20 miles per hour on a dry pavement.

2. The reflectorized symbols were identified at greater distances than were the legends on any type of sign except the interior-illuminated signs. It made little difference whether they were at 10° or 30° toward the road from a line perpendicular to the centerline or whether they were 12 or 22 feet from the centerline. The average legibility distances were about 80 feet and for only 2 out of 72 observations was the legibility distance less than 50 feet. Signs reflectorized with glass buttons were only slightly more effective than those with the reflectorized coatings used.

3. The exterior-illuminated signs were not legible for as great a distance as a number of the best reflectorized signs when both types were located 12 feet from the centerline. Since it made little difference where the exterior-illuminated signs were located, their legibility distances and ratings compared favorably with the best reflectorized signs when both were 22 feet from the center of the roadway. At greater distances, the exterior-illuminated signs would no doubt be the more legible.

Although about half of the observers read the four-letter legends at distances over 50 feet, the legibility distance was 30 feet or less for 21 percent of the observations, and, therefore, exterior-illuminated signs would not be effective at locations where a driver would be required to bring his vehicle to a stop from a speed of 20 miles an hour at a point adjacent to the sign after reading the legend. Where an immediate stop would not be required or where the sign could be located in advance of the stop, the legibility distance would be satisfactory. As with the interior-illuminated signs, the exterior-illuminated signs also have a target value, that is, they can be seen beyond the range of the headlamp beam. This feature, which was much more pronounced for the interior- than exterior-illuminated signs, may result in decreased speeds prior to reaching the point where the signs are legible.

It may be possible to increase the legibility distance for exterior-illuminated signs by using a flat finish. However, if this were required to obtain satisfactory legibility, one of the chief advantages of an exterior illuminated sign would be lost, namely, being able to utilize existing signs by attaching a simple light fixture.

4. Of the reflectorized signs, those with reflecting buttons were more satisfactory than those with reflectorized coatings when placed either 12 or 22 feet from the center of the roadway. This is not entirely consistent with the results obtained on the test route (table 2) where the legibility ratings were higher for the signs with reflectorized materials than for the signs with reflector buttons. The difference in either case, however, is not great. Since the signs were placed so that more direct comparisons could be made during the latter series of tests, these results (table 6) are more conclusive than the results for the test route (table 2). Final decision as to which type of sign to install should therefore, probably be based on the availability of materials and the particular local conditions involved. The average legibility distance

TABLE 6.—*Legibility distances of signs at various angles with the roadway observed from vehicles moving 15 to 20 miles per hou*

Color and material used			Angle with line perpendicular to roadway	For 2-lane road—12 feet from center of road								For 4-lane road—22 feet from center of road								
Letters	Panel	Background		Number of observations	Maximum legibility distance			Distribution of legibility ratings			Rating factor [1]	Number of observations	Maximum legibility distance			Distribution of legibility ratings				Rating factor [1]
					Maximum	Minimum	Average	Good	Fair	Poor			Maximum	Minimum	Average	Good	Fair	Poor	Seen	
			Degrees		*Ft.*	*Ft.*	*Ft.*	*Pct.*	*Pct.*	*Pct.*			*Ft.*	*Ft.*	*Ft.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	
GROUP 1.—BACKGROUND OF MATERIAL HAVING A REFLECTORIZED COATING																				
Black paint		Silver R. C. type A	30	6	70	35	48	83	17	0	92	6	80	30	49	17	67	16	0	56
Do		White R. C. type A	10	16	75	35	52	50	50	0	75	8	60	0	24	13	25	12	50	42
Do		do	30	20	75	30	52	50	45	5	74	12	75	0	38	17	41	25	17	50
Do		do	45	14	60	25	41	0	64	36	44	11	40	0	15	7	14	22	57	36
Do		White R. C. type C	30	6	60	30	43	50	33	17	72	6	30	0	7	0	17	17	66	31
GROUP 2.—LETTERS OF MATERIAL HAVING A REFLECTORIZED COATING																				
Silver R. C. type A	Black	White paint	10	6	75	40	57	33	67	0	66	6	40	0	18	0	0	50	50	29
Do	do	do	30	6	75	30	47	67	33	0	84	6	70	30	47	17	50	33	0	53
Do	do	Yellow paint	30	6	70	40	49	67	33	0	84	6	60	0	35	50	17	17	16	68
White R. C. type A	do	White paint	30	6	70	30	46	67	33	0	84	6	60	25	39	34	33	33	0	62
GROUP 3.—LETTERS REFLECTORIZED WITH GLASS BUTTONS																				
Buttons on black paint		White paint	30	6	75	40	53	83	17	0	92	6	40	0	26	0	0	67	33	31
Do		Yellow paint	30	6	100	40	64	83	17	0	92	6	80	0	51	17	50	17	16	52
Buttons on white paint	Black	White paint	10	14	100	40	71	79	21	0	90	14	110	0	66	43	36	14	7	68
Do	do	do	30	14	100	50	71	93	7	0	96	6	100	30	71	83	17	0	0	92
Do	do	Yellow paint	30	6	90	40	57	100	0	0	100	6	75	0	41	17	50	17	16	52
GROUP 4.—EXTERIOR-ILLUMINATED SIGNS																				
Black paint		White paint	10	16	75	20	46	12	56	32	51	8	60	15	47	12	63	25	0	52
Do		do	30	8	75	20	51	25	63	12	60									
Do		Yellow paint	10	8	80	30	51	25	38	37	56	8	90	30	59	25	63	12	0	61
GROUP 5.—INTERIOR-ILLUMINATED SIGNS																				
Opal glass		Black paint	10	16	150	70	116	100	0	0	100	8	150	50	108	100	0	0	0	100
GROUP 6.—REFLECTORIZED CROSSROAD SYMBOLS																				
Silver R. C. type A		None	10	14	110	40	75	86	14	0	93	14	100	50	78	71	29	0	0	86
Do		do	30	8	100	40	58	88	12	0	94									
Buttons on white paint		do	10	8	110	50	74	75	25	0	88	8	100	60	79	88	12	0	0	94
Do		do	30	14	120	55	84	100	0	0	100	6	110	55	81	67	33	0	0	84

[1] Obtained by adding all of the percentage rated "good," one-half the percentage rated "fair," one-third the percentage rated "poor," and one-fourth the percentage rated "seen".

of a white sign with a black panel and reflector buttons in white letters was about 70 feet. Better legibility was obtained, especially, for the 4-lane highway condition, when the sign was mounted at an angle of 30° than at an angle of 10° toward the roadway from a line perpendicular to the centerline. The legibility distance when the sign was mounted at 30° was never less than 40 feet for the 2-lane highway condition and not less than 30 feet for the 4-lane highway condition.

5. The legibility of signs made with materials having reflectorized coatings was better, especially under 4-lane conditions, when the signs were placed at 30° toward the roadway from a line perpendicular to the centerline than when placed at either 10° or 45°. There was little difference between 10° and 30° for the 2-lane condition. With the same type of material, the results were about the same whether the background or letters were reflectorized. However the materials used had a marked effect on legibility. Reflectorized signs with the silver colored type A material were legible from an average distance of about 50 feet under either the 2- or 4-lane condition. To a few observers, some signs with reflectorized coatings were not legible from a distance of more than 30 feet for a 2-lane highway and except for the one material, were not legible to the majority of observers when placed 22 feet from the center of the roadway.

HEADLIGHT BEAM CHARACTERISTICS DISCUSSED

One of the most noticeable factors influencing the legibility distances of the various reflectorized signs was the variation in the height of the headlight beam at any distance from the car. Even though the surface was exceptionally smooth, the light beam was continually varying in height especially at the greater distances from the car. Even a slight acceleration or deceleration caused a considerable variation; in fact, enough so that it was more desirable to accelerate immediately after a sign was noticed than to decelerate (without applying brakes) in order to read the sign.

The vertical motion of the headlight beam was one of the primary reasons for greater legibility distance for the symbols and signs reflectorized with glass buttons as compared to the signs having reflectorized coatings. No sign or symbol was visible until illuminated by the light beam. However, the symbols could be identified and the legends on the signs reflectorized with buttons read with only intermittent

Figure 10.—Pattern of Blackout Head Lamp Beam on Vertical Surface.

illumination, while the signs with reflectorized coatings were not legible until the vehicle was at least close enough to throw a continuous beam of light on the legend.

The effect of sign placement, including angularity, height above the road surface, and distance from the centerline of the roadway, and the effect of the type of reflecting materials on the legibility distance, can best be explained by a study of the headlight beam characteristics.

Figure 10 shows a blackout headlamp pattern on a vertical surface. It may be noticed that the top of the area has a very sharp cut-off and that the area just below the top and near the center is brighter than other portions of the pattern. At a distance of 10 feet from the lamp, the bright area is about 5 feet wide while the entire illuminated area is about 12 feet wide. Vertically, the illuminated area does not vary in intensity to any marked degree until about 5 inches below the top cut-off, then the intensity decreases until the illuminated area fades out 30 inches below the top cut-off. There was some variation in the illuminated areas for individual lights used during the road tests but the major difference was that the top of the illuminated area outside the bright area curved sharply downward in some cases and remained nearly horizontal in others. With the headlight adjusted so that the visual cut-off slopes down at the rate of 2½ inches in 10 feet (the lowest rate found to be practical in preventing the beam from shining above the vertical when the cars were traveling along a smooth, level road), the theoretical height of the beam at various distances ahead of a vehicle was as follows:

Distance from vehicle, feet	*Height of beam above road surface, inches*
0	44
20	39
40	34
60	29
80	24
100	19
120	14
140	9
160	4
176	0

Actually, however, as a vehicle traveled over a level highway, the beam varied in height. The variation was enough to cause intermittent illumination of the top of a sign legend 21 inches above the roadway at distances exceeding 120 feet, and usually the legend was not continuously illuminated until the vehicle was within 70 feet of the sign.

The bottom of the high-intensity area of the light beam hit the road surface about 60 feet ahead of the vehicle while the bottom of the beam fell about 15 feet ahead of the vehicle. The high-intensity area increased in width 5 feet every 10 feet from the vehicle and the entire illuminated area increased in width about 12 feet every 10 feet. However, the maximum widths of both were limited by the curvature of the top of the beam. Signs for blackout purposes should therefore be mounted as low and as close laterally to the pavement edge as practicable. Unless the legend of a reflectorized sign is within 18 inches of the elevation of the crown of the roadway, the sign will be of little value and practically worthless if above 24 inches. A difference of 6 inches in the mounting height will usually make a difference of about 24 feet in the legibility distance of the best reflectorized signs.

It is also important when reflectorized signs are used that the head lamp be properly mounted on the vehicle and that the signs be turned at an angle toward the road. The following are a few factors governing the intensity of the light reflected back to the driver of a vehicle:

1. The amount of light falling on the sign and the distance the driver is from the sign.
2. The particular characteristics of the reflecting material.
3. The angle between a line perpendicular to the face of the sign and a line from the head lamp to the sign (entrance or incidence angle).
4. The angle between the driver's line of sight to the sign and a line from the head lamp to the sign (divergence angle).

HEAD LAMP SHOULD BE MOUNTED NEAR DRIVER'S LINE OF SIGHT

Since the beam from the black out head lamp is very narrow, the amount of light falling on a sign will vary greatly with the distance to the sign from the head lamp and with its vertical and horizontal position along the roadway.

Although the characteristics of reflecting materials vary greatly, the intensity of the reflected light for any particular material generally decreases as both the entrance angle and divergence angles increase. Table 7 shows how these two angles vary for two different headlight mounting positions and a number of roadway conditions. For condition A the headlight is mounted as on the vehicles used for the road tests while for condition B an assumed mounting position is used. For both conditions the driver's eyes were 54 inches above

Table 7.—*Effect of head lamp mounting location and width of road on mounting angles and materials used for reflectorized signs*

Distance from vehicle, feet	Sign 12 feet from centerline of roadway		Sign 22 feet from centerline of roadway	
	Condition A [1]	Condition B [2]	Condition A [1]	Condition B [2]
ANGLE SIGNS MUST BE PLACED TO BE PERPENDICULAR TO HEAD LAMP BEAM (ZERO ENTRANCE ANGLE)				
	° ′	° ′	° ′	° ′
100	5 06	5 43	10 43	11 19
50	10 07	11 19	20 44	21 48
30			32 14	33 42
20	24 02	26 34		
ANGLE BETWEEN LINE FROM DRIVER'S EYES TO SIGN AND LINE FROM HEAD LAMP TO SIGN (DIVERGENCE ANGLE)				
	° ′	° ′	° ′	° ′
100	0 24	1 03	0 47	1 18
50	1 05	2 25	2 12	3 24
30			5 01	6 36
20	5 16	8 01		

[1] Head lamp mounted 44 inches above roadway and 11 inches to left of centerline of vehicle.

[2] Head lamp mounted 36 inches above roadway and 24 inches to left of centerline of vehicle.

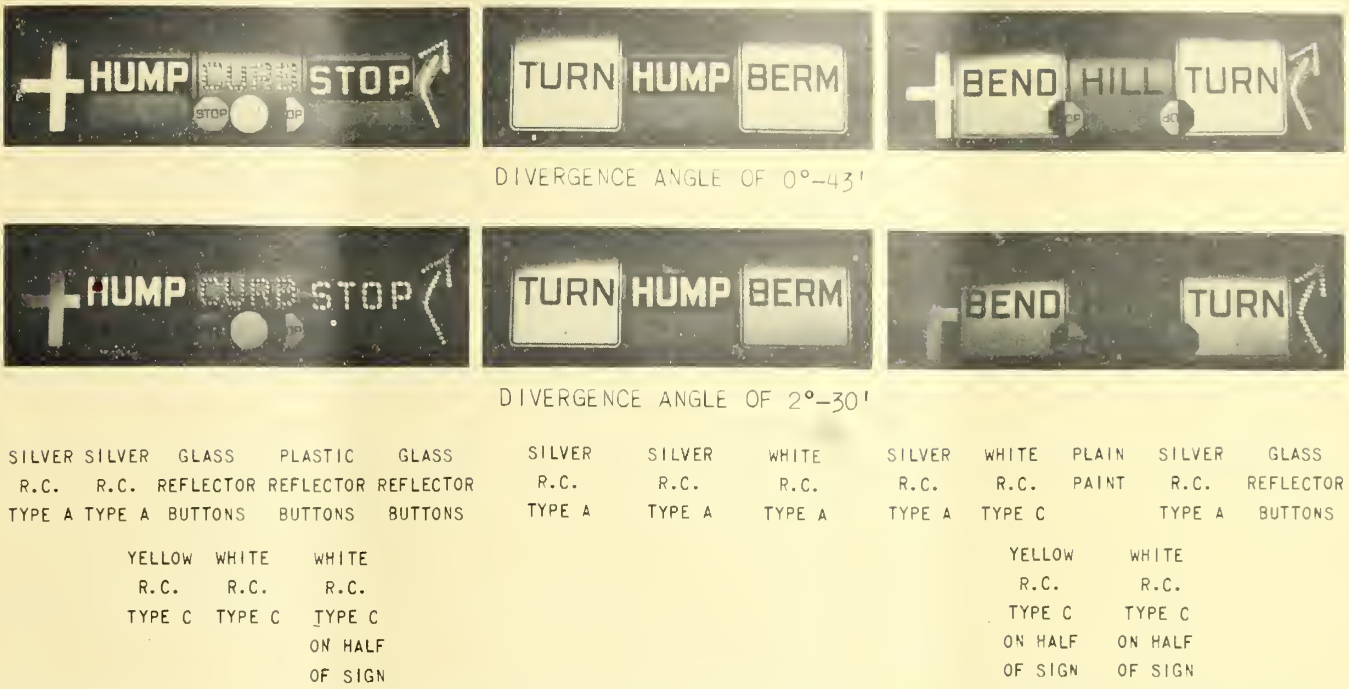


Figure 11.—Effect of Divergence Angle on Visibility and Legibility of Reflectorized Signs and Symbols Illuminated by Blackout Head Lamp. (Same Exposure Time Used in all Pictures.)

the pavement surface, 16 inches to the left of the centerline of the vehicle, and 7.5 feet behind the head lamp. The centerline of the vehicle was assumed to be 4 feet to the right of the center of the roadway (under blackout conditions, regardless of the width of the roadway, drivers try to keep just to the right of the centerline marking).

Values are shown for the 2-lane highway condition with the sign 100, 50, and 20 feet ahead of the vehicle and for the 4-lane condition with the sign 100, 50, and 30 feet ahead of the vehicle (10-foot lanes are assumed with the sign 2 feet beyond the edge). One hundred feet represents the maximum distance ahead of a vehicle that the headlight can be expected to illuminate a sign while 50 feet represents the minimum distance that a sign for use during black outs should be legible. The minimum distances at which enough light will fall on a sign to make it legible are 20 and 30 feet on 2- and 4-lane roadways, respectively.

Considering that there is a relatively small decrease in the brilliance of most reflectorized materials or glass reflecting buttons as the entrance angle increases from 0° to 10° as compared to the decrease in brilliance with a similar change at greater angles, it appears from table 7 and the results shown in table 6 that reflectorized signs should be mounted at about 15° with a line perpendicular to the centerline on a 2-lane roadway and at 25° on a 4-lane roadway.

Compared to the entrance angle, the divergence angle is much more critical. To be effective for use on signs a glass reflecting button or reflectorized material must be highly retrodirective, that is, it must return the light in approximately the same direction as the source of light. It is therefore common for the brilliance of a reflecting button to decrease more than 50 percent when the divergence angle increases from 1° to 2° (see fig. 11).

The importance of mounting the black out head lamp as near the driver's line of sight as possible is illustrated by the divergence angles for the two conditions shown in table 7. At distances over 50 feet ahead of the vehicle, the difference in the divergence angle due to the head lamp mounting location was greater than the change in the divergence angle caused by placing the sign 10 feet farther from the roadway. The values shown on table 7 also indicate that for signs that must be legible at least 50 feet away the best reflecting buttons or reflectorized materials are the ones that are most brilliant within entrance angles less than 10° and divergence angles in the neighborhood of 1° or 2°.

The answers on the 43 questionnaires filled out by the drivers and observers after they had been over the test route indicate that the illuminated signs were not superior to good reflectorized signs and that there was a slight preference for the signs reflectorized with glass buttons as compared with those made with reflectorized materials. In reply to the question "Is a self-illuminated sign superior to a good reflectorized sign?" 32 percent of the observers answered in the affirmative and 68 percent in the negative. To the question "Were the floodlighted signs adequate?" 42 percent of the observers answered "yes" and 58 percent "no." Fifty-four percent preferred the reflecting button signs and 46 percent the signs of materials with reflectorized coatings.

VISIBLE CENTERLINE MOST USEFUL MARKING IN BLACKOUTS

Seventy-seven percent of the drivers and observers felt that a visible centerline was the most useful highway marking during black out conditions. Table 8 shows the distribution of the visibility and effectiveness ratings and comparative rating factors for the 18 different types of center and edge lines studied. Most of the test route had an old centerline marking. To approach the "no line" condition, shown as the first two items in table 8, the existing line was covered with a paint as nearly as possible the same color as the pavement surface, but this marking was evidently visible to a few of the drivers.

TABLE 8.—*Visibility or effectiveness of centerline markings*

Type of line						Sky condition	Number of observations	Distribution of visibility or effectiveness ratings			Rating factor[1]
Plain or beaded	Color	Width	Solid or dashed	Length				Good	Fair	Poor	
				Dashes	Spaces						
		Inches		*Feet*	*Feet*			*Percent*	*Percent*	*Percent*	
No line						Dark	58	9	9	82	14
						Moon	18	0	5	95	3
						Total	76	7	8	85	11
Do.[2]						Dark	10	0	60	40	30
						Moon	1	0	0	100	0
						Total	11	0	55	45	28
Plain—Old line	White	4	Solid			Dark	11	0	0	100	0
						Moon	4	0	0	100	0
						Total	15	0	0	100	0
Plain	Black	4	do			Dark	24	21	21	58	32
						Moon	11	18	18	64	27
						Total	35	20	20	60	30
Do	White	4	do			Dark	60	8	10	82	13
						Moon	27	11	0	89	11
						Total	87	9	7	84	13
Do.[2]	do	4	do			Dark	20	35	10	55	40
						Moon	16	19	6	75	22
						Total	36	28	8	64	32
Do	do	6	do			Dark	50	14	16	70	22
						Moon	18	6	17	77	15
						Total	68	12	16	72	20
Do.[2]	do	6	do			Dark	24	0	17	83	9
						Moon	9	0	56	44	28
						Total	33	0	27	73	14
Plain	do	6	Dashed	20	20	Dark	21	5	19	76	15
						Moon	8	13	25	62	26
						Total	29	7	21	72	18
Beaded[2]	do	3	Solid			Dark	22	73	18	9	82
						Moon	12	67	33	0	84
						Total	34	71	23	6	83
Do.[2]	do	3	Dashed	10	25	Dark	11	9	55	36	37
						Moon	11	36	18	46	45
						Total	22	23	36	41	41
Do	do	4	Solid			Dark	48	75	12	13	81
						Moon	20	70	15	15	78
						Total	68	74	13	13	81
Do.[2]	do	4	do			Dark	15	80	7	13	84
						Moon	7	100	0	0	100
						Total	22	86	5	9	89
Do	Yellow (no passing)	4	do			Dark	34	53	23	24	65
						Moon	22	50	18	32	59
						Total	56	52	21	27	63
Do.[2]	do	4	do			Dark	21	62	33	5	79
						Moon	10	60	20	20	70
						Total	31	61	29	10	76
Beaded edge lines only	White	4	do			Dark	24	63	25	12	76
						Moon	11	73	18	9	82
						Total	35	66	23	11	78
Beaded center and edge lines	do	4	do			Dark	25	84	4	12	86
						Moon	10	100	0	0	100
						Total	35	89	3	8	91
Beaded	do	4	Dashed	10	[illegible]	Dark	20	45	30	25	60
						Moon	10	50	40	10	70
						Total	30	47	33	20	64
Do	do	4	do	10	20	Dark	21	43	28	29	57
						Moon	9	44	45	11	67
						Total	30	43	34	23	60
Do	do	4	do	20	[illegible]	Dark	24	58	13	29	65
						Moon	10	60	20	20	70
						Total	34	59	15	26	67

TABLE 8.—*Visibility or effectiveness of centerline markings*—Continued

Type of line: Plain or beaded	Type of line: Color	Type of line: Width	Type of line: Solid or dashed	Length: Dashes	Length: Spaces	Sky condition	Number of observations	Distribution of visibility or effectiveness ratings: Good	Distribution of visibility or effectiveness ratings: Fair	Distribution of visibility or effectiveness ratings: Poor	Rating factor[1]
		Inches		*Feet*	*Feet*			*Percent*	*Percent*	*Percent*	
Beaded	White	4	dashed	15	25	Dark	26	58	23	19	70
						Moon	11	64	27	9	78
						Total	37	60	24	16	72
Do.[2]	do	4	do	10	10	Dark	10	40	40	20	60
						Moon	5	60	20	20	70
						Total	15	47	33	20	64
Do.	do	4	do	10 (white)	30 (black)	Dark	22	63	23	14	75
						Moon	9	67	33	0	81
						Total	31	64	26	10	77

[1] Obtained by adding all of the percentage rated "good" to one-half the percentage rated "fair."

[2] On bituminous surface. Others on concrete surface.

Based on the data in table 8 and answers on questionnaires filled out by the drivers and observers after completing a trip over the test course, the conclusions with respect to center and edge lines for blackout conditions may be summarized as follows:

1. Visible centerlines are the most useful marking and are essential.

2. Plain painted centerlines, not reflectorized, are of little value. Plain white centerlines, including those 6 inches in width, could not be seen by many of the drivers. Out of 44 observers, 33 stated that the ordinary white centerline was inadequate. On a concrete surface, a black centerline was rated as more effective than a plain white centerline.

3. White continuous centerlines either 3 or 4 inches wide and reflectorized with glass beads are very effective. Any greater width is unnecessary.

4. Yellow reflectorized lines are not as visible as white reflectorized lines, although the difference is not great.

5. Dashed centerlines are not as effective as continuous centerlines although they are adequate on tangent sections of highway (fig. 12). Out of 44 observers, 26 stated that the continuous line was definitely better, 13 stated that the continuous line was somewhat better, and 5 had no preference. When a dashed line is used, the dashes should be at least 10 feet long and the spaces not over 20 feet long. Although both designs require the same amount of paint, 10-foot dashes and spaces are preferable to 20-foot dashes and spaces, especially on curves. However, the remarks made by the observers indicate that a solid centerline should be used in place of a dashed centerline even on fairly flat curves.

6. Edge lines when used in conjunction with a centerline (fig. 12) are effective in helping to outline the road surface but are unnecessary and may be confused with lane lines. The comments of the 44 observers regarding the edge lines were as follows:

Comment	*Number of observers*
Good if used with centerline	16
Good	11
Confusing or dangerous	9
Confusing or dangerous without centerline	2
Some help to driving	4
Unnecessary	2

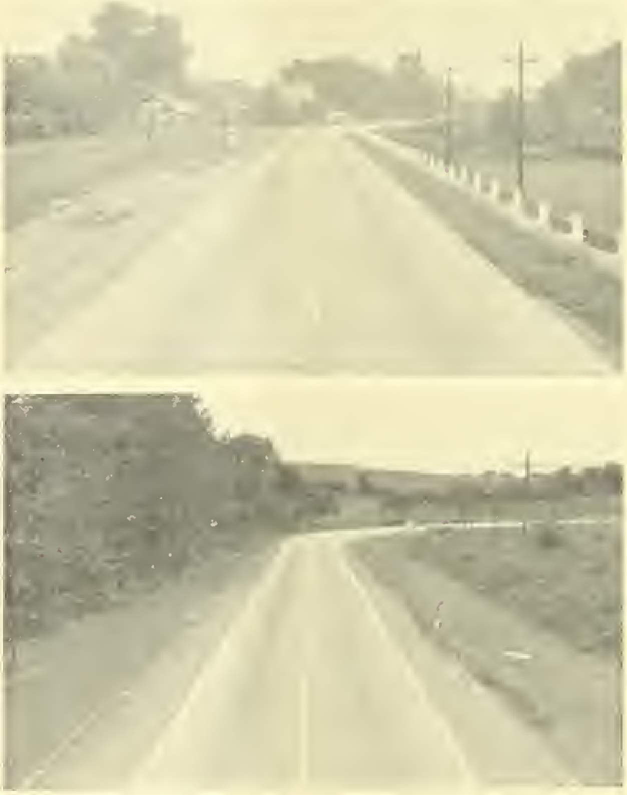


FIGURE 12.—TOP, 4-INCH BEADED WHITE CENTERLINE WITH 20-FOOT DASHES AND 20-FOOT SPACES; BOTTOM, 4-INCH BEADED WHITE CENTERLINE AND EDGE LINES.

A few drivers did mistake the 2-lane pavement with edge lines for a 3- or 4-lane pavement and drove off the paved surface. However, at some locations, such as short access roads that are narrow and have poor alinement, the use of edge lines in addition to a centerline is desirable. Most drivers exceeded a speed of 20 miles per hour while on the sections with edge lines. Some believed edge lines would make it possible to travel at speeds up to at least 30 miles per hour with the blackout head lamp.

With the exception of the 6-inch line, the visibility of the plain painted centerlines was lower when there was some moonlight than when there was no moonlight, but moonlight increased the visibility of the beaded white centerlines. However, in either case the difference was slight.

The need for certain transverse pavement markings is increased during blackout conditions. For example, at stop signs and traffic signals meeting blackout requirements some type of visible pavement marking is needed to indicate the point where the stop should be made. Under black-out conditions it is exceedingly difficult for a driver to estimate the distance to a light, and the curbs and other objects normally used to define or outline an intersection are not visible. Unless there is some special marking to indicate an intersection, a driver may pass through the intersection without being aware of it. When transverse pavement lines such as stop lines are used, they generally require special treatment in addition to reflectorization or they will not be effective.

Table 9 shows the rating factors for a number of different transverse pavement markings, some being designs that as yet have not been used extensively for normal conditions. Since it was not possible to place all the transverse markings at similar locations, the rating factor for a particular marking was influenced to some extent by the drivers' speed when the marking was noticed and also affected to some extent by other pavement markings or signs at the intersection.

TABLE 9.—*Visibility of painted crosswalk and special limit lines*

Color	Width	Kind	Sky condition	Number of observations	Distribution of visibility and effectiveness ratings			Rating factor [1]
					Good	Fair	Poor	
	Inches				*Percent*	*Percent*	*Percent*	
White	8	Plain	Dark	15	25	8	67	29
			Moon	6	50	0	50	50
			Total	21	31	6	63	34
Do.	12	do	Dark	15	33	17	50	42
			Moon	6	40	20	40	50
			Total	21	36	18	46	45
Do	8	Beaded	Dark	30	86	14	0	93
			Moon	12	100	0	0	100
			Total	42	89	11	0	95
Do	[2] 8	do	Dark	30	75	25	0	88
			Moon	12	100	0	0	100
			Total	42	86	14	0	93
Do.	[2] 8	Plain	Dark	30	64	36	0	82
			Moon	12	83	17	0	92
			Total	42	71	29	0	86
Do	[3] 8	Beaded	Dark	15	82	18	0	91
			Moon	6	100	0	0	100
			Total	21	87	13	0	94
Do.	[4] 8	do	Dark	15	70	30	0	85
			Moon	6	100	0	0	100
			Total	21	75	25	0	88
Do.	[5] 8	do	Dark	15	70	30	0	85
			Moon	6	100	0	0	100
			Total	21	79	21	0	90

[1] Obtained by adding all of the percentage rated "good" to one-half the percentage rated "fair."

[2] With beaded limit line 24 inches wide placed 4 feet in advance of crosswalk line.

[3] With 4-inch beaded lines 5 feet long spaced 24 inches apart perpendicular to crosswalk line on the side of approaching traffic to form a "comb" effect.

[4] Special design of limit line consisting of a continuous V design, the open end of each V being 3 feet wide and the height being 4 feet. One end of the design was bordered by the 8-inch crosswalk line and the other end by a similar 8-inch white beaded line.

[5] With 4-inch beaded limit line having perpendicular beaded lines toward approaching traffic 5 feet long spaced 24 inches apart to form a "comb" effect and placed 4 feet in advance of crosswalk line.

LINES HAVING "COMB" DESIGN FOUND MOST EFFECTIVE TRANSVERSE MARKING

Considering the particular locations where these transverse markings were placed, the design with 4-inch lines 5 feet long spaced 2 feet center-to-center and placed perpendicular to a transverse line on the side of approaching traffic to give a "comb" effect (fig. 13) was the most effective transverse marking. Where it is necessary that the pavement marking be visible for 50 feet such as at wide, important intersections where there is a stop light, this design should be used. Increasing the length of the perpendicular bars will, to some extent, increase the visibility distance.

At the intersection of 2-lane highways where there is a blackout traffic signal or at any intersection where there is a stop sign visible for 50 feet or more, an 8-inch beaded line is sufficiently visible to indicate the stopping point since in either case the driver will have decreased his speed below 20 miles per hour before reaching the line. An 8-inch beaded line is also sufficiently visible for identifying intersections where there is apt to be little or no cross traffic.

Painting the curbs is another means of indicating to a driver that he is at an intersection and is of material assistance to the driver when making a turn but does not provide an advance warning so that a stop can be made before entering the intersection. Since the vertical faces of curbs are low, they are illuminated at greater distances than other surfaces and therefore are the most effective location for markings. Curbs in the direct line of traffic whether at intersections or around hazardous objects should be marked.

Table 10 shows the visibility ratings for the painted and reflectorized curbs along the test route. None of the observers noticed curbs at one intersection that had been painted yellow. The rating factor for the plain white curbs is nearly as high as that for the beaded curbs but is misleading since the rating factors in this table are based only on the adjective ratings given by the observers that noticed the painted curbs. Only 67 percent of the observers noticed the plain white curbs while all observers noticed the beaded curbs. If an adjective rating of "poor" were assumed each time the plain white curb was not seen, the rating factor would be

TABLE 10.—*Visibility of painted curbs at intersections*

Color	Kind	Sky condition	Number of observations	Distribution of visibility ratings			Rating factor [1]
				Good	Fair	Poor	
				Percent	*Percent*	*Percent*	
White	Plain	Dark	45	57	29	14	72
		Moon	18	83	0	17	83
		Total	63	65	20	15	75
Yellow	do	Dark	8	0	0	100	[2] 0
White	Beaded	Dark	15	71	29	0	86
		Moon	6	50	50	0	75
		Total	21	67	33	0	84
White & black	do	Dark	45	69	31	0	85
		Moon	18	67	17	16	76
		Total	63	69	26	5	82
Do. (straight curb at T-intersection).	do	Dark	30	91	9	0	96
		Moon	12	190	0	0	100
		Total	42	94	6	0	97

[1] Obtained by adding all of the percentage rated "Good" to one-half the percentage rated "Fair."

[2] None of the 8 observers saw this marking.

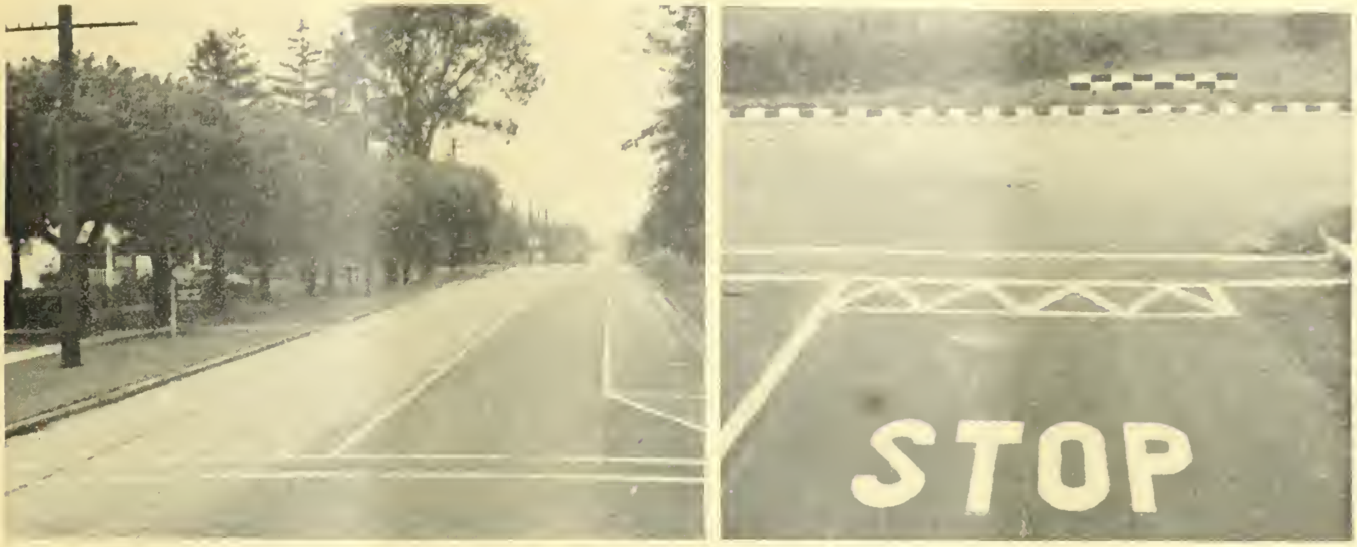


FIGURE 13.—BEADED WHITE PAVEMENT MARKINGS IN URBAN AREAS. LEFT, CENTERLINE, PARKING STALLS, AND "COMB" DESIGN AT CROSSWALK; RIGHT, PAVEMENT AND CURB MARKINGS AT T-INTERSECTION.

reduced from 75 to 40. The white curbs were, therefore, less than half as effective as the beaded curbs. The white beaded curbs with plain black vertical stripes were no more effective than the white beaded curbs.

At the railroad crossing within the limits of Hebron the standard railroad crossing marking was painted on the pavement surface with plain white paint for half the roadway width and with reflectorized paint for the other half. The comb design was also employed to emphasize the approach lines parallel with the tracks on the side that was reflectorized (see fig. 3). The reflectorized markings were rated as about 60 percent more effective than those made using plain white paint (table 11).

Parking stall lines if visible may be effective in helping prevent emergency vehicles from hitting parked cars especially when there are cars near the middle of a block but none near the intersections. However, their use for this purpose is very limited since, when the stalls are filled, the transverse lines will not be visible. Plain painted stalls were practically useless (table 10) unless the marking included a longitudinal line parallel with the curb (fig. 13) and even then they were only about half as effective as the beaded markings.

To warn a driver of the approach to an intersection on a 2-lane road an additional line may be placed alongside the centerline for 100 feet in advance of the intersection or when a dashed centerline is used it may be made continuous for 100 feet in advance of the intersection. Such markings are very effective (table 11). On 4-lane roads with a double centerline marking, the same principle may be applied by the use of an additional line near the centerline and by making lane lines solid instead of dashed. Double lines should never be used for lane lines since with the blackout light such a lane line on the left side of the lane may be mistaken for the centerline marking.

The only words marked on the pavement were located where the test drivers had reduced their speed in compliance with a good reflectorized or illuminated sign. At these locations, both beaded and plain painted words were fairly effective. Where vehicle speeds are not reduced below 20 miles per hour, words probably will not be effective. Their use is therefore very limited.

TWO TYPES OF ROAD-EDGE DELINEATORS TESTED

A traffic signal equipped with a 27-volt transformer was in operation during 2 of the 3 nights when the tests were made. It was very effective and probably more noticeable than are lights under normal conditions operating on 110 volts.

Some type of road-edge delineation is desirable at curves on unpaved roadways where the centerline cannot be marked, and necessary at sharp curves even though the centerline of the roadway is marked. The beam of the blackout lamp is so limited that practically no light falls on the centerline at sharp curves especially when the vehicle is on the outside of the curve. Two types of markers suitable for use as delineators were tested. One was constructed by wrapping 3-inch boards for 42 inches with a reflectorized material (fig. 14–A) and the other consisted of three sets of three plastic reflecting buttons mounted 1 foot apart on a 3-inch board. When these markers were installed in a vertical position 5 feet outside the pavement edge, the reflectorized material extended from 6 to 36 inches above the pavement elevation and the centers of the three sets of reflecting buttons were at 12, 24, and 36 inches above the pavement.

The outside of four horizontal curves and the tangents for about 80 feet in advance of and beyond the curves were delineated using the markers with reflecting buttons. The spacing was 40 feet on two curves, 20 feet on one curve, and 80 feet on the other curve during the first night that the drivers' reactions were recorded. Bidirectional units were used on one curve. On the second night, alternate markers on each curve were removed so that results for two different spacings would be obtained for the same curve. The markers with reflectorized material were placed individually along the route.

Conclusive data as to the most desirable spacing on curves were not obtained. However, three delineators were visible at all times only when spaced 40 feet on the more gradual curves (3° to 5°) and 20 feet on the one sharp curve (about 12°). The sets of buttons mounted 36 inches above the pavement were rarely visible, the sets 24 inches above the pavement were visible about half the time, and those at a height

TABLE 11.—*Visibility and effectiveness of various white urban pavement markings and blackout traffic signal*

RAILROAD GRADE CROSSING MARKINGS

Legend or marking	Kind	Width of line	Sky condition	Number of observations	Distribution of visibility and effectiveness ratings			Rating factor [1]
					Good	Fair	Poor	
		Inches			*Percent*	*Percent*	*Percent*	
Railroad crossbuck [2]	Plain	Standard [3]	Dark	15	50	25	25	63
			Moon	6	20	60	20	50
			Total	21	33	45	22	56
Do.[2]	Beaded	do.[3]	Dark	15	100	0	0	100
			Moon	6	100	0	0	100
			Total	21	100	0	0	100
Lines parallel to and 10 feet from tracks	do	12	Dark	15	50	42	8	71
			Moon	6	25	75	0	63
			Total	21	44	50	6	69
Do.[4]	do	12	Dark	15	100	0	0	100
			Moon	6	100	0	0	100
			Total	21	100	0	0	100

WHITE PAINTED PARKING STALL LINES

Legend or marking	Kind	Width of line	Sky condition	Number of observations	Good	Fair	Poor	Rating factor [1]
7-foot transverse lines	Plain	4	Dark	15	0	0	100	0
			Moon	6	50	0	50	50
			Total	21	12	0	88	12
Do	Beaded	4	Dark	15	80	20	0	90
			Moon	6	0	100	0	50
			Total	21	67	33	0	84
7-foot transverse lines with longitudinal line.	Plain	4	Dark	15	40	0	60	40
			Moon	6	33	33	34	50
			Total	21	38	12	50	44
Do	Beaded	4	Dark	15	86	0	14	86
			Moon	6	50	0	50	50
			Total	21	78	0	22	78

WHITE PAINTED LANE LINES EXTENDING 100 FEET BACK FROM INTERSECTIONS

Legend or marking	Kind	Width of line	Sky condition	Number of observations	Good	Fair	Poor	Rating factor [1]
	Beaded	4	Dark	15	88	12	0	94
			Moon	6	75	25	0	88
			Total	21	83	17	0	92

WHITE PAINTED WORDS ON PAVEMENT

Legend or marking	Kind	Width of line	Sky condition	Number of observations	Good	Fair	Poor	Rating factor [1]
Stop	Plain		Dark	15	57	29	14	72
			Moon	6	0	100	0	50
			Total	21	44	45	11	67
Slow [2]	Beaded		Dark	16	50	50	0	75

BLACKOUT TRAFFIC SIGNAL

Legend or marking	Kind	Width of line	Sky condition	Number of observations	Good	Fair	Poor	Rating factor [1]
			Dark	7	100	0	0	100
			Moon	6	100	0	0	100
			Total	13	100	0	0	100

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[1] Obtained by adding all of the percentage rated "good" to one-half the percentage rated "fair."

[2] With 5-foot letters. Crossbuck 20 feet long and 8 feet wide.

[3] As given in Manual on Uniform Traffic Control Devices for Streets and Highways, by the A. A. S. H. O.

[4] With 4-inch beaded lines 5 feet long and 24 inches center to center perpendicular to line parallel to tracks on approach side to form a "comb" effect.

of 12 inches were visible all the time when they were within the light beam and not more than 120 feet ahead of the vehicle. When one set of three reflecting units is used on delineators, the set should be within 18 inches of the elevation of the road surface. However, one reflecting button at 12 inches, one at 18 inches, and one at 24 inches would probably be more effective especially on curves with considerable superelevation or where there also is a vertical curve.

The rating factor for the units wrapped with material having a reflectorized coating was not as high as for the units with plastic reflecting buttons (table 12) mainly because they were at individual locations and were approached unexpectedly. To be effective enough to warrant their installation as delineators, the reflectorized surface facing traffic should be at least 2 inches wide and 24 inches high with the bottom as close to the

(Continued on page 135)

VOLCANIC CINDERS SUITABLE FOR USE IN BASE COURSE CONSTRUCTION

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by **EDWARD A. WILLIS**, Associate Highway Engineer, **HENRY AARON**, Associate Highway Engineer, and **RICHARD C. LINDBERG**, Junior Highway Engineer

Volcanic cinders, a fragmental ash deposited as a result of volcanic eruptions, has been used for many years in constructing and maintaining local earth roads. Recently it has been used in the construction of road surfaces, base courses, and subbases on State highway systems.

A survey was recently made to obtain information on the service behavior of volcanic cinder roads in Arizona and to correlate the road condition with the physical characteristics of the cinders as determined by laboratory tests. Laboratory and circular track tests were made on cinders from five sources in Arizona and on "tufa" gravel.

The results of these studies disclose that volcanic cinders will prove satisfactory for use as base courses for thin bituminous surface treatments if they meet, with certain modifications, the requirements of A. A. S. H. O. specification M–56–38 for type C stabilized base-course materials. Even though the most satisfactory cinders are nonplastic, adequate compaction and density of base courses can be obtained by the judicious use of water during rolling.

"Tufa" gravel was not satisfactory as a base course because of an excess of fine-size material. "Tufa" gravel with a coarser gradation would probably serve satisfactorily as a base material. The investigation provided a direct correlation between circular track tests and field service behavior. It established the fact that suitable base-course materials will withstand concentrated traffic in the circular track with water 2½ inches above the top of the subbase but that water 4½ inches above the top of the subbase provides a condition more severe than can reasonably be expected under normal service conditions.

VOLCANIC CINDERS have been used for many years in the construction and maintenance of local earth roads in the southwestern United States. More recently this material has been utilized in the construction of road surfaces, base courses, and subbases on State highway systems. Since it is available in practically unlimited quantities in certain areas, its utilization in road building has considerable economic importance.

This report describes (1) a survey made by the Division of Tests in 1939 in cooperation with the Arizona State Highway Department and the Phoenix office of the Public Roads Administration to obtain information on the service behavior of roads constructed with volcanic cinders and to correlate the road condition with the physical characteristics of the cinders as determined by laboratory tests, and (2) laboratory and circular track tests on cinders from five sources and a "tufa" gravel, all selected by engineers in the Public Roads administration office at Phoenix in collaboration with the materials department of the Arizona Highway Department. One of the cinder materials tested in the circular track was similar to that used in the construction of Federal-aid project No. 105–C, surveyed in 1939. This provided a direct correlation between the track tests and observed service performance.

The cinders consist of a fragmental ash deposited in cone-shaped mounds or ridges as a result of volcanic eruptions. Figures 1–A and 1–B show a number of such cones and ridges in the vicinity of Flagstaff, Arizona. The fragments, ranging in size from fine sand grains to large boulders, have a honeycomb structure and are relatively light in weight. The material occurs as a compact deposit in the undisturbed condition and the sides of excavations will stand vertically, but as soon as it is dislodged it crumbles into a loose mass which is difficult to recompact. The character of the material, both undisturbed and dislodged, is shown in figures 1–C and 1–D.

The cinders vary considerably in color, both in the same deposit and in different deposits. Various shades of yellow, red, purple, and black are encountered. They all have practically the same granular texture, honeycomb structure and, unless mixed with clay, they are nonplastic. Small amounts of a calcareous material, known locally as caliche, are often present.

The deposits are generally covered with from 4 to 10 feet of overburden consisting of a friable, grayish silty soil containing rock fragments and calcareous material or a brownish red to red plastic clay in the upper part and red clay mixed with partially decomposed cinders in the lower part passing gradually into the unweathered cinders.

CONDITION SURVEY MADE OF ROADS IN SERVICE

The condition survey was made during April 1939. Samples taken at that time were tested in the Public Roads laboratory.

The roads covered in this survey are located in the vicinity of Showlow and Springerville in the east central part of the State and near Flagstaff in north central Arizona. They pass through areas of rolling topography at elevations exceeding 7,000 feet and are subject to heavy snowfall during the winter.

A total of about 83 miles of road were inspected, of which approximately 20 miles were of the bituminous surface-treated type and the remainder was untreated. The untreated surfaces were maintained in uniformly good condition as illustrated in figure 2–A. Since no variation in condition could be observed, no detailed studies were made of the untreated roads.

Bituminous treatments were used on FAP 105–C and FAP 95–J located, respectively, on U. S. 60 west of Springerville and on U. S. 89 north of Flagstaff. Failures were observed on short sections of FAP 105–C. FAP 95–J was in good condition throughout its length. Both sections were constructed in 1937.

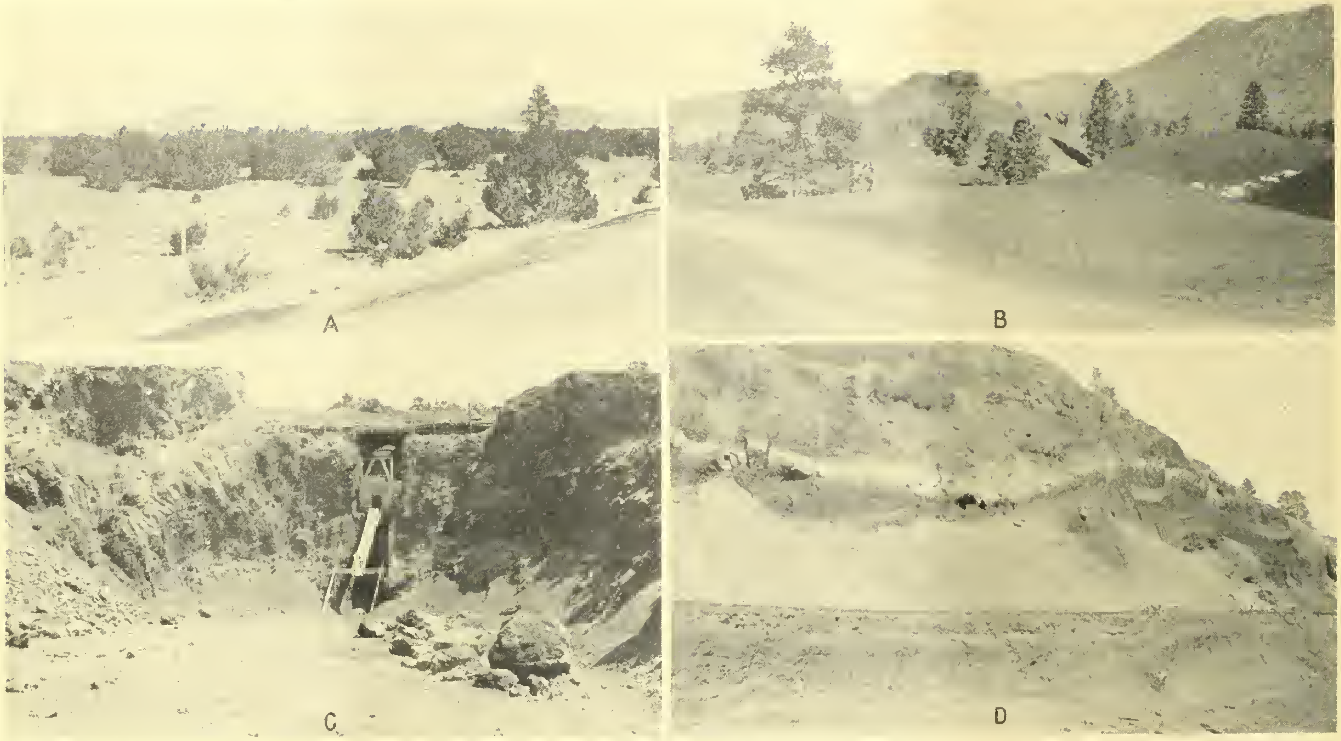

Figure 1.—Volcanic Cinder Deposits in Arizona. Cinder Cones North of Flagstaff (A and B); Cinder Pit East of Showlow (C), and West of Flagstaff (D).

The bituminous surface treatment consisted of from 0.75 to 1.00 gallon of liquid asphalt (SC–2) applied to a previously sprinkled, rolled, and properly shaped surface. No cover material was spread unless the asphalt was not entirely absorbed. The loose material developed in blading the road during the reshaping process was used for the cover material when required. The asphalt was placed in two applications, the first consisting of 0.5 gallon per square yard and the second varying from 0.25 to 0.50 gallon. The character of the surface obtained in this manner is illustrated in figures 2–B and 2–C.

The base course referred to in the discussion of service performance is the layer of cinders on which the asphaltic surface treatment was applied. This layer was about 5 inches thick on the two projects surveyed. The penetration of the bituminous material had formed a mat approximately 1 inch thick. Figure 3 shows a section through the mat and base course.

The base course rested on a cinder sub-base consisting of pit run material designated "imported borrow." The depth of this material was not uniform as it was varied during construction in accordance with the subgrade soil encountered. In many instances, shallow fills from 2 to 3 feet high were composed entirely of cinders.

Excellent drainage was provided in all cases. The cross-sections and grades were adequate to remove any surface water and there were no indications of subsurface water.

The surveying and sampling consisted of (1) making a condition survey of the bituminous surface, (2) measuring the thickness of the base and subbase, (3) examining the subgrade soils, and (4) obtaining samples of the base course, sub-base, and subgrade from locations corresponding to the variations in road condition.

Samples were obtained from the base and subbase after removing the portion penetrated by bituminous material. In all cases care was taken not to include any subbase material with the base-course samples, and likewise not to include any subgrade with the subbase samples.

The thickness of base and subbase was measured and the subgrade soil was examined at each sampling point. Each type of subgrade encountered was sampled.

For convenience of analysis, the results of the condition survey and the mechanical analyses and physical tests performed on the base-course samples are summarized in table 1. The base thickness shown in this table includes the portion penetrated by the bituminous material, which amounted to approximately 1 inch. The characteristics of the subbase and subgrade material are shown by the test results given in tables 2 and 3, respectively.

Three variations in surface condition were encountered. They have been designated (1) intact, (2) cracked and scaled, and (3) badly cracked and rutted. Figures 2–B and 2–C are views of the surface condition corresponding to the designation "intact." Cracked and scaled surface is illustrated in figure 4–B and a badly cracked and rutted section is shown in figure 4–A.

The similarity of the subbase cinders on both projects is illustrated in table 2. They were granular, nonplastic materials and, referring to table 1, it will be seen that they compare favorably in grading with the base-course materials on which no failures occurred. Table 1 also shows that the thickness of subbase was just as great or greater under the failed areas as under the intact areas.

The subgrade soils were stony loams and sandy loams containing calcareous material in various amounts The results of tests shown in table 3 indicate that the soils had physical properties of both the A–2 and A–5

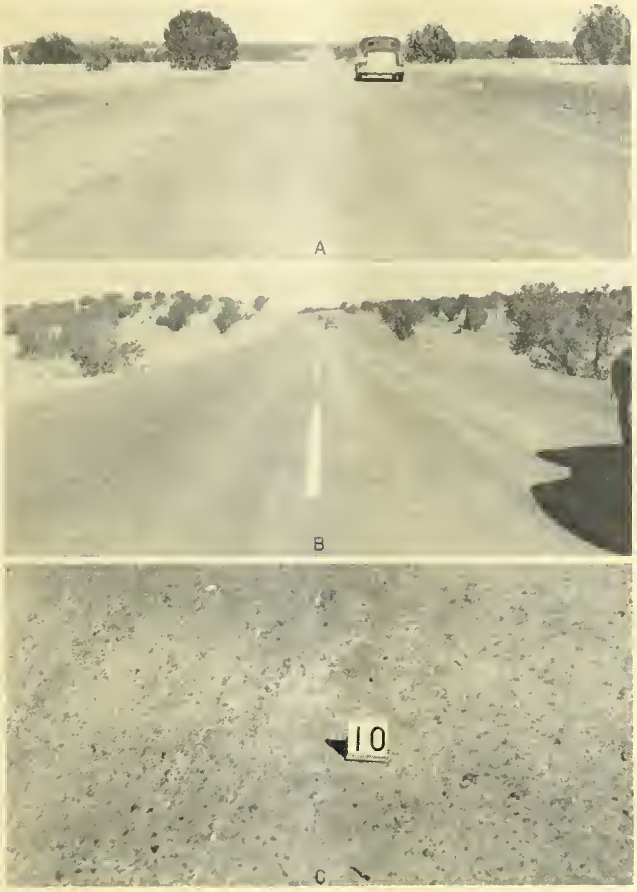

FIGURE 2.—VOLCANIC CINDER ROADS. A, UNTREATED CINDER ROAD TYPICAL OF ALL SUCH ROADS INSPECTED; B, BITUMINOUS TREATED CINDER ROAD IN GOOD CONDITION (FAP 95–J); C, TEXTURE OF ROAD SURFACE SHOWN IN B.

subgrade groups. The same type of subgrade was encountered under both the intact sections and the locations where failures occurred.

The base course was uniformly 5 inches thick except at the cracked and scaled location where base-course sample S–12653 was obtained. The 13 inches recorded at this point probably consisted of 5 inches of base course and 8 inches of subbase, but no difference in character of the material could be detected for the full depth. The subgrade at this location was the same as that where base-course sample S–12647 was taken and where the total thickness of base and subbase equalled 14 inches.

The good condition of all but a few short sections indicated that the base-course thickness of 5 inches was adequate for the traffic carried on these projects provided the base-course materials were satisfactory. Since the failed areas were interspersed between the intact sections, both the character and volume of traffic on the intact portions were the same as on the sections where cracking, scaling, and rutting occurred. The average daily traffic on the two projects as counted in 1937 and 1940 is given in table 4.

ROAD CONDITION RELATED TO GRADATION AND PHYSICAL PROPERTIES OF BASE COURSES

The results of the condition survey and laboratory tests summarized in table 1 indicate that the difference in road condition is directly related to the character of the base-course materials. The base-course samples taken from the locations where no failures occurred were nonplastic and had liquid limits not exceeding 30. In contrast, the samples from the failed sections had plasticity indexes of 7 and 14 and corresponding liquid limits of 33 and 37.

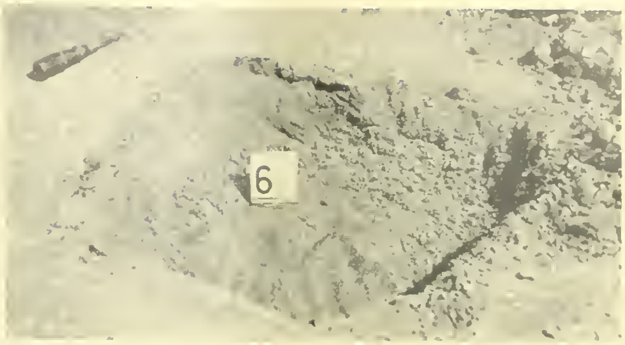

FIGURE 3.—SECTION THROUGH BITUMINOUS PENETRATION MAT AND BASE COURSE. BOTTOM OF EXCAVATION IS TOP OF SUBBASE.

TABLE 1.—*Summary of results of condition survey and mechanical analysis and physical tests performed on base course samples*

F. A. project No	105–C	105–C	105–C	105–C	105–C	95–J	95–J
Sample No.	S–12647	S–12650	S–12653	S–12654	S–12656	S–12657	S–12659
Surface condition	Intact	Intact	Cracked scaled	Badly cracked, rutted	Intact	Intact	Intact
Thickness:							
Base ..... inches	5	5	13	5	5	5	5
Subbase ..... do	9	14	[1] 0	[2] 15+	[3] 0	[2] 18+	11
Grading:							
Passing 1½-inch sieve ..... percent	100	100	100	100	100	100	100
Passing 1-inch sieve percent	96	100	97	100	100	98	100
Passing ¾-inch sieve percent	82	93	94	99	99	95	99
Passing ⅜-inch sieve percent	70	84	86	90	91	91	95
Passing No. 4 sieve percent	61	74	66	76	73	77	85
Passing No. 10 sieve percent	53	62	57	58	53	61	74
Passing No. 40 sieve percent	45	50	47	50	39	44	64
Passing No. 200 sieve percent	18	19	29	32	21	21	18
Analysis of material passing No. 10 sieve:							
Sand, 2.0–0.25 millimeter ..... percent	16	20	18	13	26	28	14
Sand, 0.25–0.05 millimeter ..... percent	57	53	37	37	40	41	67
Silt ..... do	21	17	32	35	28	23	9
Clay ..... do	6	10	13	15	6	8	10
Colloids ..... do	2	5	2	4	0	0	2
Dust ratio [4]	40	38	62	64	54	48	28
Tests on material passing No. 40 sieve:							
Liquid limit	30	30	33	37	29	24	20
Plasticity index	[5] NP	[5] NP	7	14	[5] NP	NP	NP
Shrinkage limit	24	25	20	16	25	20	
Shrinkage ratio	1.6	1.6	1.7	1.8	1.6	1.7	
Centrifuge moisture equivalent	23	22	28	28	19	19	10
Field moisture equivalent	31	31	31	28	31	26	25
Specific gravity	2.76	2.72	2.73	2.72	2.74	2.79	2.87
Subbase sample No. [6]	S–12648	S–12651		S–12655		S–12658	S–12660
Subgrade sample No. [7]	S–12649	S–12652	S–12649				S–12661

[1] Base and subbase of same material.

[2] Sampled on fill section. Entire fill composed of cinders. Figures indicate depth sampled.

[3] No subbase. Fill of coarse rock fragments and loamy soil similar to subgrade sample No. S–12652, table 3.

[4] Dust ratio $=100\left[\frac{\text{percentage passing No. 200 sieve}}{\text{percentage passing No. 40 sieve}}\right]$.

[5] NP is used to denote nonplastic materials which do not have a plastic limit, in contrast to those which have a plastic limit equal to the liquid limit and therefore a plasticity index of zero.

[6] Refers to samples in table 2.

[7] Refers to samples in table 3.

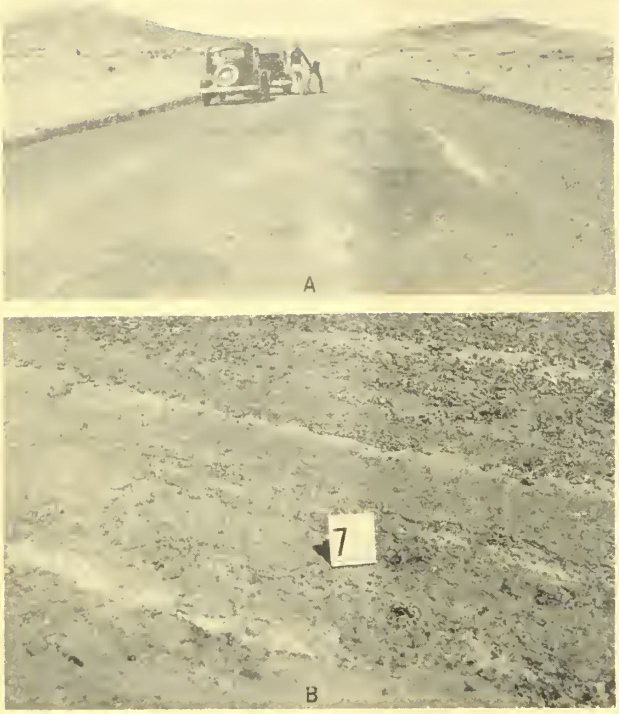

Figure 4.—A, Failed Section of FAP 105–C That was Badly Cracked and Rutted; B, Cracked and Scaled Portion of FAP 105–C.

Table 2.—*Results of mechanical analyses and physical tests performed on subbase samples*

F. A. project No.	105–C	105–C	105–C	95–J	95–J
Sample No.	S–12648	S–12651	S–12655	S–12658	S–12660
Grading:					
Passing 1½-inch sieve percent	100	88	80	100	100
Passing 1-inch sieve do	99	88	72	98	100
Passing ¾-inch sieve do	95	86	68	97	99
Passing ⅜-inch sieve do	86	84	58	94	94
Passing No. 4 sieve do	70	74	47	82	89
Passing No. 10 sieve do	56	61	38	63	83
Passing No. 40 sieve do	40	43	31	49	71
Passing No. 200 sieve do	14	16	8	21	14
Analysis of material passing No. 10 sieve:					
Sand, 2.0–0.25 millimeter percent	29	29	19	23	14
Sand, 0.25–0.05 millimeter do	53	49	64	49	71
Silt do	13	15	13	24	10
Clay do	5	7	4	4	5
Colloids do	2	2	2	2	1
Dust ratio[1]	35	37	26	43	20
Tests on material passing No. 40 sieve:					
Liquid limit	22	24	21	27	18
Plasticity index	[2] NP	[2] NP	[2] NP	[2] NP	[2] NP
Shrinkage limit		23		24	
Shrinkage ratio		1.6		1.6	
Centrifuge moisture equivalent	10	16	7	19	9
Field moisture equivalent	31	31	34	33	28
Specific gravity	2.78	2.71	2.78	2.75	2.87

[1] Dust ratio$=100\left[\frac{\text{percentage passing No. 200 sieve}}{\text{percentage passing No. 40 sieve}}\right]$.

[2] NP is used to denote nonplastic materials which do not have a plastic limit, in contrast to those which have a plastic limit equal to the liquid limit and therefore a plasticity index of zero.

With respect to gradation the samples from the failed areas had 29 and 32 percent passing the No. 200 sieve as compared to a maximum of 21 for the samples from the locations where no failures occurred. No significant difference existed in the coarser fractions of the samples from the various locations.

In general, the survey showed that there were cinders from widely separated sources eminently suitable for road construction and indicated that it would be possible to select materials by means of the mechanical analysis and plasticity tests. However, since there are many other types of cinders available which had not been used in road construction, it was considered advisable to conduct a series of tests on such materials in the circular track with the particular purpose of determining whether they were satisfactory for use as base courses under a thin bituminous surface treatment. Accordingly, a sufficient quantity of cinders from each of five sources was procured. A quantity of material known locally as a "tufa" gravel with sand was also submitted and included in the tests.

Table 3. *Results of mechanical analyses and physical tests performed on subgrade samples*

F. A. project No.	105–C	105–C	95–J
Sample No.	S–12649	S–12652	S–12661
Grading:			
Passing 1½-inch sieve percent	84	81	56
Passing 1-inch sieve do	81	79	50
Passing ¾-inch sieve do	79	76	47
Passing ⅜-inch sieve do	75	68	43
Passing No. 4 sieve do	69	63	39
Passing No. 10 sieve do	63	57	35
Passing No. 40 sieve do	60	49	30
Passing No. 200 sieve do	39	30	14
Analysis of material passing No. 10 sieve:			
Sand, 2.0–0.25 millimeter do	4	14	15
Sand, 0.25–0.05 millimeter do	44	41	53
Silt do	33	25	24
Clay do	19	20	8
Colloids do	5	6	2
Dust ratio[1]	65	61	47
Tests on material passing No. 40 sieve:			
Liquid limit	44	42	35
Plasticity index	1	12	3
Shrinkage limit	21	22	26
Shrinkage ratio	1.7	1.6	1.6
Centrifuge moisture equivalent	30	30	24
Field moisture equivalent	33	36	34
Specific gravity	2.71	2.70	2.76

[1] Dust ratio$=100\left[\frac{\text{percentage passing No. 200 sieve}}{\text{percentage passing No. 40 sieve}}\right]$.

Table 4.—*Average daily traffic on Federal-aid projects 105-C and 95-J for the years 1937 and 1940*

Project No.	Average daily traffic			
	1937		1940	
	Total	Trucks and busses	Total	Trucks and busses
105–C	95	25	125	38
95–J	210	35	474	64

"Tufa" gravel is a porous concretionary formation of calcium carbonate deposited frequently around springs. The material submitted, however, was not calcareous and is in reality a pumiceous tuff, a product of volcanic action and very similar to the cinders. Since it is known locally as "tufa" gravel that designation has been retained throughout this report.

In each case, all material larger than 1 inch was removed, crushed to pass a 1-inch sieve and recombined with the fraction passing the 1-inch sieve. The identification and source of the six materials tested in the track are shown in table 5.

The circular track used in this investigation was the same as that used in the studies of water-retentive chemicals as admixtures with nonplastic road-building materials which have been reported previously.[1] The

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[1] Studies of Water-Retentive Chemicals with Nonplastic Road-Building Materials, by E. A. Willis and C. A. Carpenter. PUBLIC ROADS, vol. 20, No. 9, November 1939.

TABLE 5.—*Identification and source of materials tested in circular track*

Circular track section No.	Description and source
1	"Tufa" gravel with sand. From pit 0.5 mile right of station 48, FAP 81-B.
2	Red cinders. From pit 1.0 mile right of station 235, F. H. 17-B2.
3	Black cinders. From pit 600 feet left of station 2695 south approach road to Grand Canyon National Park.
4	Red cinders. From pit 600 feet right of station 881, FAP 89-D.
5	Red cinders. From pit 1,500 feet right of station 1909, FAP 105-C. This project inspected and surveyed in 1939.
6	Red cinders. From pit 2,200 feet right of station 431, F. H. 30-B.

tire equipment was size 30×5 of the high-pressure type, requiring an inflation pressure of 80 pounds per square inch.

Distributed traffic, which was used for compacting the base course and the surface treatment, was obtained by gradually shifting the rotating beam longitudinally with respect to its axis of rotation. Concentrated traffic, which was used after the surface treatment had been constructed, was obtained by locking the sliding pivot of the beam in such a position that the wheels pursued two concentric courses whose centerlines were about 2½ inches on either side of the centerline of the test sections.

Each of the six sections tested in the track was 18 inches wide, 6 inches deep, and approximately 6.3 feet long. All the sections were laid over a crushed stone subbase through which water introduced from below could pass. The grading and soil constants of the 6 materials tested in the circular track are given in table 6.

TABLE 6.—*Grading and soil constants of materials tested in circular track*

	Track section No.—					
	1	2	3	4	5	6
Sieve analysis of total sample:						
Passing 1-inch sieve……percent	100	100	100	100	100	100
Passing ¾-inch sieve……do	100	99	100	100	99	98
Passing ⅜-inch sieve……do	100	90	89	96	95	87
Passing No. 4 sieve……do	98	85	68	83	81	78
Passing No. 10 sieve……do	92	81	43	60	60	64
Passing No. 40 sieve……do	67	59	17	33	34	39
Passing No. 200 sieve……do	40	24	9	19	19	12
Dust ratio [1]	60	41	53	58	56	31
Analysis of material passing No. 10 sieve:						
Sand, 2.0-0.25 millimeter……percent	34	31	67	53	49	53
Sand, 0.25-0.05 millimeter……do	30	40	17	20	26	33
Silt……do	28	18	12	17	18	10
Clay……do	8	11	4	10	7	4
Colloids……do	1	2	1	3	2	1
Physical constants of material passing No. 40 sieve:						
Liquid limit	26	35	17	23	18	22
Plasticity index	[2] NP	[2] NP	[2] NP	[2] NP	[2] NP	[2] NP
Shrinkage limit	23	-	-		-	-
Shrinkage ratio	1.6	-		-		-
Centrifuge moisture equivalent	14	18	13	17	14	6
Field moisture equivalent	28	40	18	27	24	29
Specific gravity	2.64	2.58	2.98	2.96	2.78	2.80

[1] Dust ratio $=100\left[\frac{\text{Percentage passing No. 200 sieve}}{\text{Percentage passing No. 40 sieve}}\right]$.

[2] NP = Nonplastic.

In constructing the test sections sufficient water was mixed with the dry material so that, when squeezed in the hand, a firm ball would be formed. Mixing was continued to distribute the moisture thoroughly. The dampened materials were then placed in the track in two approximately equal layers. The bottom layer was first tamped into place with hand tamps and then compacted with 1,000 wheel-trips of distributed traffic. The top layer was then placed and hand tamped, and compaction with distributed traffic was continued until a total of 20,000 wheel-trips was completed.

The compacted base was made ready for a surface treatment by trimming the sections smooth and priming with 0.3 gallon of light tar. After curing, the surface treatment was constructed. It consisted of 0.4 gallon of hot bituminous material and a cover of 50 pounds per square yard of ¾-inch maximum size stone. This surface treatment was consolidated by 20,000 wheel-trips of distributed traffic. At the end of this period the surface was well sealed and showed no movement.

DIFFICULTY EXPERIENCED IN COMPACTING SECTION 3

The behavior of the materials was judged on the basis of the appearance of the sections at various stages of the test, supplemented by measurements of vertical displacement of the surface. Previous reports have described the transverse[2] and longitudinal[3] profilometers with which the measurements were made.

The area between the initial and each succeeding transverse profile was measured by a planimeter. The measured area divided by the track width, 18 inches, gave the vertical displacement, and the average for the two stations on the section gave the average vertical displacement for that section.

The area between the initial and each succeeding longitudinal profile made in that wheel lane was measured for each section and the area of vertical displacement determined. That area divided by the length of the wheel lane gave the depth of rutting and the average for the two wheel lanes gave the average depth of rutting for the section.

Figure 5 shows changes in behavior of the various sections under altered test conditions. The abrupt changes in the slopes of the displacement curves coincide with the increased severity of test conditions brought about by changing from distributed to concentrated traffic or by raising the ground-water level.

The schedule of traffic applications with notations on the behavior of the six test sections is given in table 7. Initial profile measurements were made at 40,000 wheel-trips. From 40,000 to 60,000 wheel-trips, the sections were subjected to distributed traffic, and the ordinates at 60,000 wheel-trips (fig 5) represent displacements at the end of such traffic. At 60,000 wheel-trips, concentrated traffic was begun and continued to the end of the test.

Section 1 was quite spongy during the early stages of compaction and material in the section tended to shove up onto the curbs. Light scarifying and raking was necessary to keep the section smooth. After 18,000 wheel-trips, the material in section 1 began to knit together and by 20,000 wheel-trips it was firm and stable.

Much difficulty was experienced in compacting section 3. The cinders placed in this section were moistened and subjected to the same treatment accorded all the other sections. It did not set up readily, however, and additional water was sprinkled on the surface in an effort to aid compaction. This was ineffectual The upper portion of the section was so loose that traffic had to be applied at slow speed in order to prevent the material from being thrown out of the trough Under

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[2] Circular Track Tests on Low-Cost Bituminous Mixtures, by C. A. [illegible] and J. F. Goode, PUBLIC ROADS, vol. 17, No. 4, June 1936.

[3] A Study of Sand-Clay-Gravel Materials for Base Course [illegible] C. A. Carpenter and E. A. W[illegible], PUBLIC ROADS, vol. 20, No. [illegible]

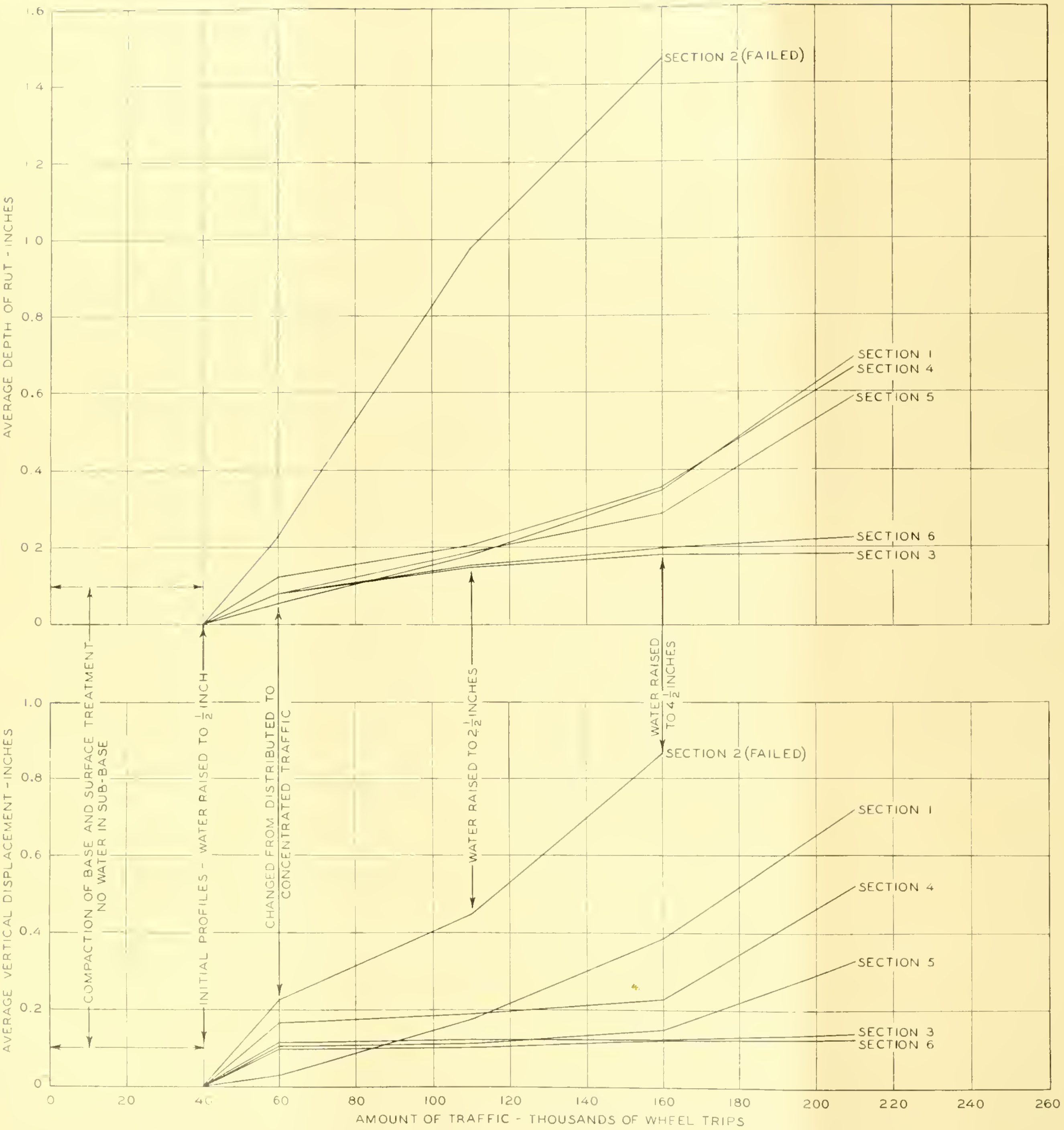

Figure 5.—Rate of Surface Displacement of Circular Track Test Sections Under Traffic.

continued traffic and as the cinders in section 3 dried out, the material gradually consolidated.

At 6,800 wheel-trips, it was possible to increase the speed of traffic to that normally used in testing. At this time the material had dried out a great deal, and compaction seemed to take place more rapidly. At 12,700 wheel-trips, section 3 was in good condition and only a very light sprinkling was necessary from time to time during the remainder of the compaction period to keep the surface from dusting and loosening. Light sprinkling was necessary also on sections 1 and 5 to prevent the surface from becoming loose.

All other sections compacted readily under distributed traffic with no water in the subbase.

The density of each track section was measured at 20,000 wheel-trips or just before the application of the surface treatment. The results of these and later determinations are shown in table 8.

Water was admitted and the level raised to ½ inch above the top of the subbase at 40,000 wheel-trips. The following 20,000 wheel-trips of distributed traffic under this condition did not develop any failures and all sections were firm and stable. Profiles taken at the end of this period showed that all sections had under-

TABLE 7.—*Schedule of operations and behavior of test sections*

Operation	Traffic	Water level above top of subbase	Behavior					
			Section No. 1	Section No. 2	Section No. 3	Section No. 4	Section No. 5	Section No. 6
	Wheel-trips	*Inches*						
Placing and compacting base	0-20,000	[1]0	Good	Good	Good	Good	Good	Good.
Compacting treated surface	20,000-40,000	[1]0	do	do	do	do	do	Do.
Testing with distributed traffic	40,000-60,000	½	do	do	do	do	do	Do.
Testing with concentrated traffic	60,000-110,000	½	Slightly unstable	Unstable	do	do	do	Do.
Do	110,000-160,000	2½	Became more unstable.	Failed	do	Slightly unstable	do	Do.
Do	160,000-210,000	4½	Unstable		do	Unstable	Slightly unstable	Do.

[1] No water in subbase.

gone further compaction. Sections 2 and 4 experienced the most compaction during this interval (see fig. 5).

TABLE 8.—*Comparison of moisture contents and densities obtained by laboratory compaction and in the track sections*

Method of compaction	Section No.	Moisture content based on weight of dry soil	Density	
			Dry weight per cubic foot	Solids by volume
		Percent	*Lb. per cu. ft.*	*Percent*
Track sections at end of initial compaction period (40,000 wheel-trips).	1	13.2	103.8	63.0
	2	14.4	61.5	38.2
	3	4.4	124.1	66.7
	4	11.5	85.0	46.1
	5	6.5	93.9	54.1
	6	6.6	97.9	56.1
Track sections at time of failure (sec. 2, 160,000 wheel-trips) or at end of test (210,000 wheel-trips).	1	16.8	108.0	65.6
	2	31.7	64.4	39.1
	3	9.8	122.2	65.6
	4	24.7	114.6	62.1
	5	18.1	92.3	53.2
	6	14.9	94.1	53.9
Standard compaction test on fraction passing No. 4 sieve.[1]	1	16.5	102.5	62.2
	2	25.2	85.5	58.1
	3	16.5	110.2	59.4
	4	21.0	100.0	54.2
	5	21.0	101.8	58.8
	6	20.4	99.8	57.1

[1] Results shown are the optimum moisture content and maximum density determined by the standard compaction test.

Testing with concentrated traffic was begun at 60,000 wheel-trips with the ground-water elevation remaining at ½ inch. At 83,680 wheel-trips movement was noticed in sections 1 and 2. The remaining sections at this time were in excellent condition. At 110,000 wheel-trips, the conclusion of testing with ground water ½ inch above the top of the subbase, the surfacing in section 2 had been ruptured in two places. At this time the surface of section 1 was in distress and rutting of the surface had begun. Profiles taken at 110,000 wheel-trips showed that sections 2, 4, and 1 had undergone the most displacement, in the order named. The rest of the sections exhibited negligible amounts of displacement between 60,000 and 110,000 wheel-trips as shown in figure 5.

Water was raised to 2½ inches above the top of the subbase at 110,000 wheel-trips and testing with concentrated traffic was resumed. At 120,000 wheel-trips, movement under the wheel loads was noticed in section 4. Increased rutting took place in sections 1 and 2. At the completion of 50,000 wheel-trips of concentrated traffic at this ground-water elevation (making a total of 160,000 wheel-trips) section 2 had failed. Profiles taken at this time showed that sections 1 and 4 had undergone much displacement during this period of the test. Figure 6 shows the condition of the sections at the conclusion of 160,000 wheel-trips.

SECTIONS 3 AND 6 REMAINED IN GOOD CONDITION THROUGHOUT SEVERE TEST

Water was temporarily drained from the subbase at this time while samples were being taken from section 2 for final density determinations. The results obtained are shown in table 8. In order to continue the testing of the five remaining sections, a bituminous cold patch material was used to level the surface of section 2. No profile measurements were made on this section after 160,000 wheel-trips.

Testing was continued with the ground-water elevation raised to 4½ inches above the top of the subbase. The severity of these test conditions soon caused sections 1 and 4 to become very rough and unstable. At 174,160 wheel-trips, section 4 had developed deep ruts and the surfacing had broken in one wheel rut, exposing the brownish-red volcanic cinder base. Figure 7–C shows the condition of this section at 210,000 wheel-trips, the end of the test.

At 174,160 wheel-trips, section 1 had practically reached a point of failure, and the tufa gravel material was exposed between the wheel ruts. At the end of 210,000 wheel-trips, section 1 had definitely failed. Figure 7–A shows the appearance of this section at the end of the test.

Section 5, which had remained in good condition previously, began to show some movement in the base. By the end of the test considerable displacement had occurred as shown in figure 7–D. The displacement curves, figure 5, show clearly that there was a marked increase in the rate of surface displacement in section 5 after the ground-water elevation had been raised to 4½ inches above the top of the subbase. Comparison of figures 6–E and 7–D show the difference in behavior of this section under altered test conditions.

Sections 3 and 6 remained in excellent condition throughout the track tests. Very little surface displacement took place after 60,000 wheel-trips (see fig. 5). Figures 7–B and 7–E show the appearance of sections 3 and 6, respectively, at the end of the tests. Comparison with figures 6–C and 6–F shows changes in appearance since that at 160,000 wheel-trips. The smooth appearance of the wheel ruts is caused by material carried from sections 1 and 2 where the surfacing had failed.

The density of each track section was measured at the time of failure or at the end of the testing period. The results of these determinations as well as those obtained after compaction are shown in table 8 together with the densities obtained in the standard compaction test on the fraction of the material passing the No. 4 sieve.

No consistent relationship was found between the densities measured by the laboratory compaction test and those obtained in the track either after the initial

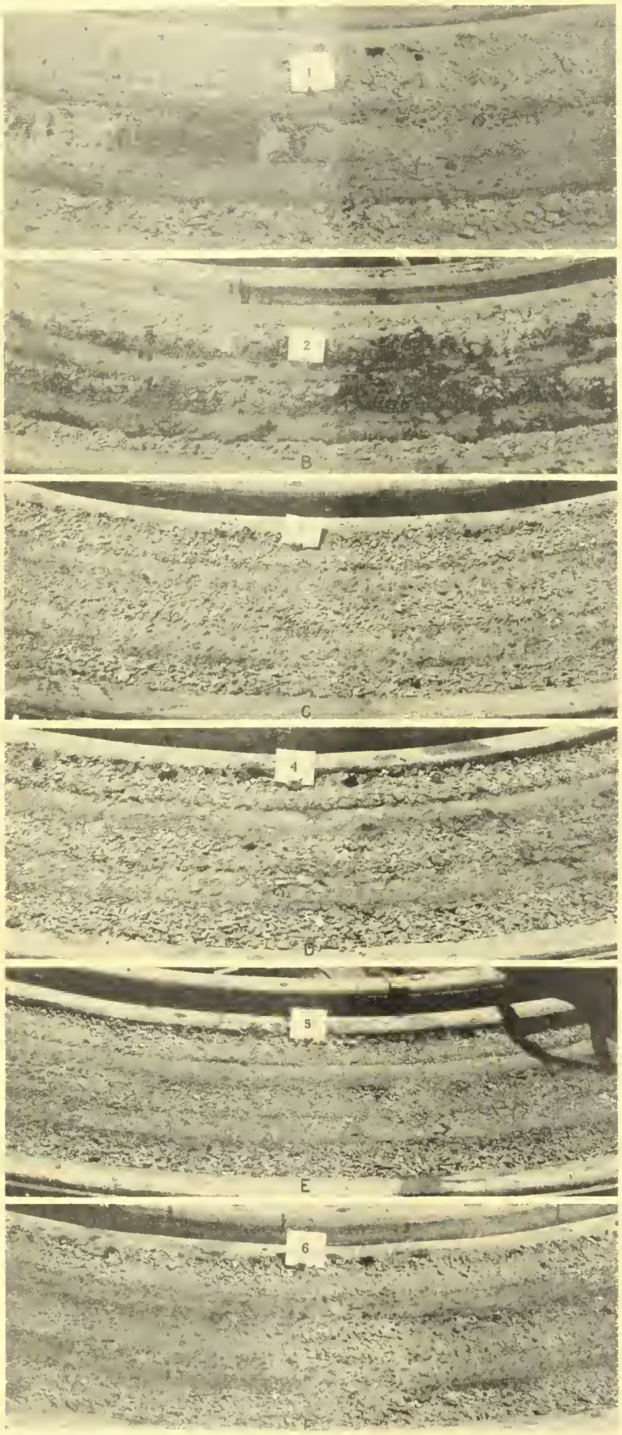

Figure 6.—Appearance of Circular Track Test Sections After 160,000 Wheel-Trips of Traffic. A, Surfacing of Section 1 Failed Between Wheel Ruts Because of Movement in Base; B, Base-Course Material of Section 2 Shoved up Through Surface Treatment Between Wheel Ruts; C, Little Displacement Occurred on Section 3, Surface Treatment in Excellent Condition; D, Some Movement Took Place in Section 4; E, Little Displacement Occurred on Section 5, Surface Treatment in Excellent Condition; F, Base and Surface of Section 6 in Excellent Condition.

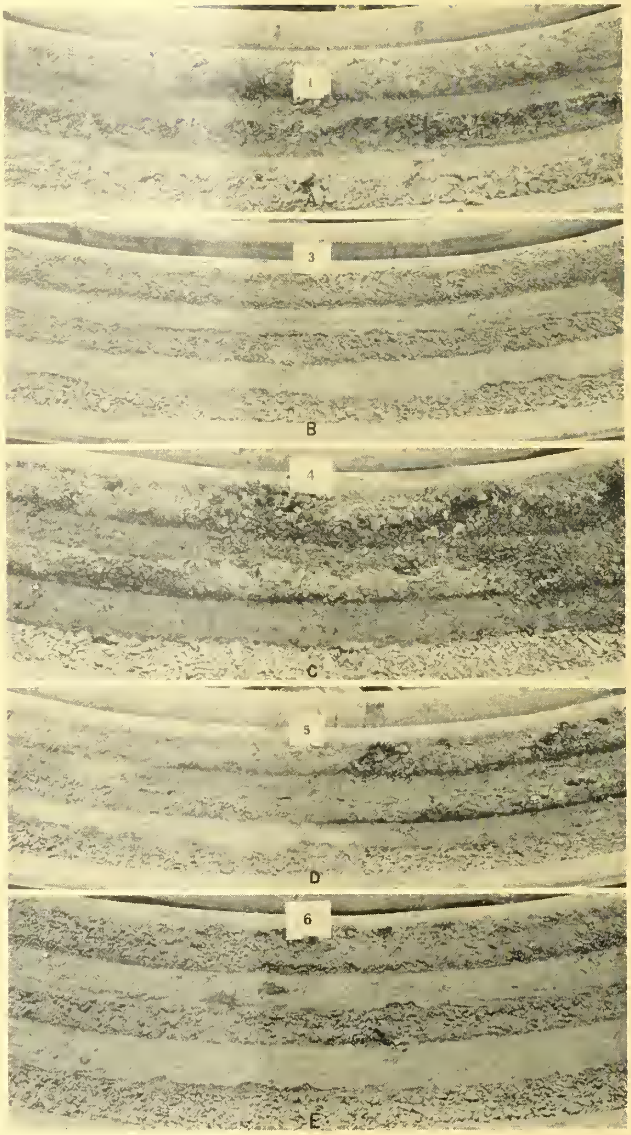

Figure 7.—Appearance of Circular Track Test Sections After 210,000 Wheel-Trips (End of Test). A, Section 1 Had Failed; Section 2 (Not Shown) Had Previously Failed; B, Section 3 Remained in Good Condition (Compare With Fig. 6–C); C, Section 4 Rutted and Surface Broke, Exposing Base; D, Considerable Displacement Occurred on Section 5 (Compare With Fig. 6–E); E, Section 6 Remained in Good Condition (Compare With Fig. 6–F).

compaction period or at the end of the test. Although profiles taken at the end of 20,000 wheel-trips of distributed traffic (interval from 40,000 to 60,000 wheel-trips) showed that appreciable compaction had taken place in all sections, the determinations made at the end of the test showed that sections 3, 5, and 6, which exhibited the least movement under traffic, were slightly less dense than at the end of the initial compaction period. It is probable that a major part of the measured displacement occurring under distributed test traffic was due to further consolidation of the surface treatment. Increases in density were noted in sections 1, 2, and 4 which failed or were unstable at the end of the test.

This investigation makes possible a direct correlation between the circular track tests and service performance since the material tested in section 5 of the track came from the same source as the cinders used in constructing FAP 105–C, a project which was included in the field condition survey.

The mechanical analyses and physical constants of the five samples taken from FAP 105-C together with the respective surface condition ratings appear in table 1. The grading and constants of the material tested in section 5 are given in table 6. The behavior of this section under test is summarized in table 7.

The laboratory tests on samples from FAP 105-C showed that the base courses in areas where no failure occurred were nonplastic, that they had liquid limits of 30 or less, and that the amount of material passing the No. 200 sieve did not exceed 21 percent. Where failures occurred, liquid limits of 33 and 37, plasticity indexes of 7 and 14, and percentages passing the No. 200 sieve of 29 and 32, were determined by tests on base-course samples.

The cinders tested in section 5 of the circular track were nonplastic. The liquid limit was 18 and the amount passing the No. 200 sieve was 19 percent. It can be inferred, therefore, that the section 5 material would have proved entirely satisfactory for use as a base course under field service conditions.

SECTION 2 ONLY MATERIAL REVEALED UNSATISFACTORY BY TRACK TEST

In reports of previous circular track investigations, it has been stated that an average vertical displacement of 0.25 inch was sufficient to cause marked damage to the bituminous wearing course. Furthermore, it was found that, with the ground water elevation ½ inch above the top of the subbase, concentrated traffic provided a condition sufficiently severe to enable the identification of the definitely unsatisfactory materials. In the present investigation and under these test conditions, section 5 gave excellent performance. Satisfactory behavior with average vertical displacements below 0.20 inch was also observed while testing with water at the 2½-inch elevation. (See table 7 and figs. 5 and 6–E).

It was not until the ground water had been raised to 4½ inches above the top of the subbase or within about 1½ inches of the surface that section 5 showed excessive movement. Since section 5 was constructed of material having known satisfactory characteristics, it is evident that the test conditions imposed near the end of the investigation were more severe than are normally encountered in service. Using a criterion that definitely unsatisfactory materials will not withstand concentrated traffic with water ½ inch above the top of the subbase without excessive vertical displacements and that wholly satisfactory materials will withstand concentrated traffic with water 2½ inches above the top of the subbase without excessive movement, the five cinders and the tufa gravel tested in the circular track are rated as shown in table 9.

Section 1 was slightly unstable with water at the ½-inch level and is rated as poor. Section 2 was definitely unstable under this condition and failed completely with water at the 2½-inch level. It is, therefore, considered unsatisfactory. Sections 3, 5, and 6 did not exhibit excessive movement with water at the 2½-inch level and are rated as satisfactory. Section 4 gave good service with water at the ½-inch level but became slightly unstable when the water was raised an additional 2 inches. It is rated as fair although it is believed that under reasonably good drainage conditions in the field it would give satisfactory service.

TABLE 9.—*Rating of sections after testing under concentrated traffic with water elevation 2½ inches above the top of the subbase*

Section No.	Composition	Service rating
1	Tufa gravel with sand	Poor.
2	Red cinders	Unsatisfactory.
3	Black cinders	Satisfactory.
4	Red cinders	Fair.
5	do.	Satisfactory.
6	do.	Do.

Figure 8 shows the grading curves for samples Nos. S–12653 and S–12654 (table 1), which were the two samples taken from points where failures had occurred on FAP 105–C, and for the materials from sections 1 and 2 which showed excessive movement in the circular track tests with water 2½ inches above the top of the subbase. The shaded band in the figure was drawn to include the grading curves of all materials investigated which had shown satisfactory performance as base courses in roads or in the circular track.

The grading curve for the tufa gravel tested in section 1 falls almost entirely outside the shaded band, being finer than any of the other materials investigated. The other three materials that gave unsatisfactory service have grading curves which fall outside the shaded band in the vicinity of the No. 200 sieve size.

Section 4 was unstable and section 5 was slightly unstable at the end of the test under the extremely severe conditions which were imposed. The materials tested in these two sections have gradings which fall entirely within the band of satisfactory materials. Likewise, the materials tested in sections 3 and 6, which gave excellent service throughout all phases of the testing, have grading curves falling within the band. An examination of the analyses of material passing the No. 10 sieve given in table 6 shows that for the section 3 and 6 materials only 4 percent of the soil mortar was clay while the clay content of section 5 was 7 percent and that of section 4 was 10 percent. Similarly, the silt plus clay contents of sections 3 and 6 were respectively 16 and 14 percent, that of section 5 was 25 percent, and that of section 4 was 27 percent. It is possible that the larger amount of very fine material present in the soil mortar fraction was responsible for the adverse behavior of sections 4 and 5 under the extremely severe testing conditions.

Samples Nos. S–12653 and S–12654 taken from failed locations on FAP 105–C had plasticity indexes of 7 and 14 respectively. All other base-course materials included in this investigation were nonplastic. This investigation did not furnish sufficient information to determine the maximum allowable plasticity index for base-course materials.

CONFORMITY OF MATERIALS WITH SPECIFICATION FOR STABILIZED BASE-COURSE MATERIALS STUDIED

A. A. S. H. O. specification M 56–38 gives requirements for materials for stabilized base course. Volcanic cinders are not included within the three types of materials covered by the specification. However, they most closely resemble the type C materials (gravel, stone or slag screenings, or sand) and a comparison of the results of tests on the cinders with the type C specification requirements is, therefore, of interest

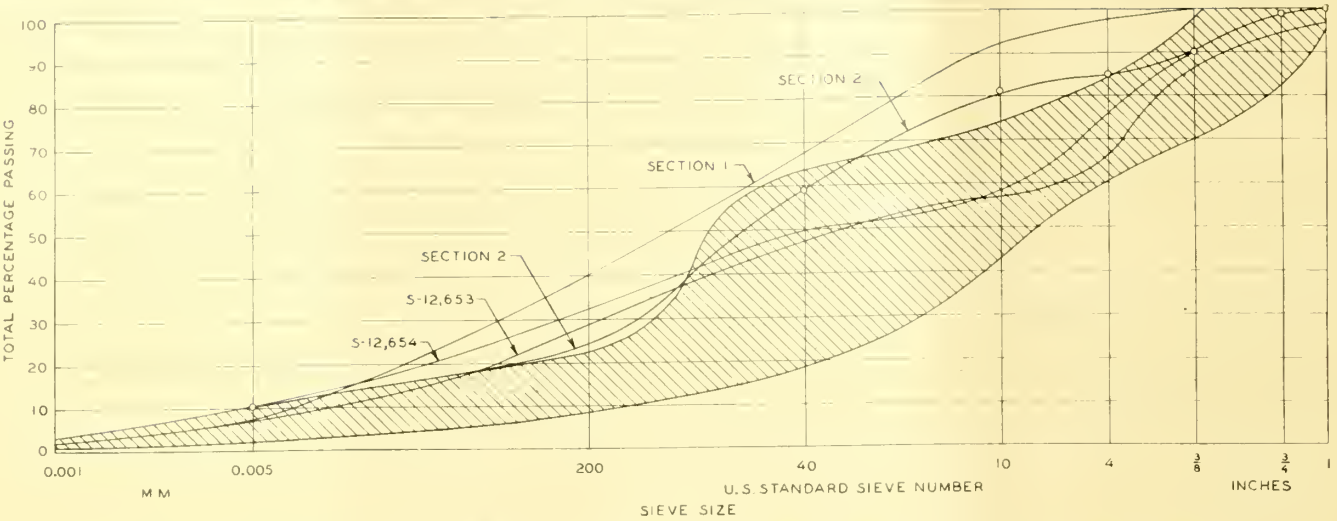

Figure 8.—Grading Band of Satisfactory Cinders and Grading Curves of Unsatisfactory Cinders.

The requirements for type C materials contained in the specification are as follows:

Passing—	*Percentage by weight*
¾-inch sieve	100
No. 4 sieve	70–100
No. 10 sieve	35–80
No. 40 sieve	25–50
No. 200 sieve	8–25

The fraction passing the No. 200 sieve shall be less than one-half of the fraction passing the No. 40 sieve. The fraction passing the No. 40 sieve shall have a liquid limit not greater than 25 and a plasticity index not greater than 3.

All of the samples from locations designated as intact in table 1 were somewhat coarser than required by the specification since in each case there was less than 100 percent passing the ¾-inch sieve. Sample No. S–12647 had only 61 percent passing the No. 4 sieve as compared to the minimum of 70 percent contained in the specification for this size. In all other respects, the samples from intact locations conformed to the grading requirements of the A. A. S. H. O. Samples No. S–12647, S–12650, and S–12656 had liquid limits greater than 25 but not greater than 30.

Samples Nos. S–12657 and S–12659 had liquid limits less than 25 as required by the specification. All five of the samples under discussion were nonplastic.

In four cases the percentage passing the No. 200 sieve was less than one-half of the percentage passing the No. 40 sieve while in the case of sample No. S–12656 this ratio was 0.54.

The samples from areas where distress was noted (table 2) also had maximum-size particles larger than is covered by the specification. These two samples, S–12653 and S–12654, failed to meet the grading requirements of the specification in another respect since the percentages passing the No. 200 sieve were 29 percent and 32 percent respectively as compared to a maximum permissible percentage of 25. The liquid limits, the plasticity indexes, and the ratios of the percentage passing the No. 200 sieve to the percentage passing the No. 40 sieve are all too high for these two samples to conform to the specification.

In the circular track tests, section 1 was rated as poor (see table 9). The material used in the construction of this section failed to meet the A. A. S. H. O. type C requirements because it had too large percentages of material passing the Nos. 10, 40, and 200 sieves, and because it had a liquid limit greater than 25 and a dust ratio greater than 0.50. Section 2 material was rated as unsatisfactory. It failed to meet the specifications because the percentages passing the No. 10 and No. 40 sieves were too large and the liquid limit was 35. Section 4 material was rated as fair and met the requirements in all respects except that the dust ratio was greater than one-half.

Sections 3, 5, and 6 were rated as satisfactory in the circular track tests. Section 3 failed to meet the specification requirements in that it had too small an amount of material passing the No. 4 and No. 40 sieves and had a dust ratio of 0.53. The cinders used in sections 5 and 6 had a small amount of material retained on the ¾-inch sieve but otherwise met the specification except that the section 5 material had a dust ratio slightly greater than one-half.

CONCLUSIONS

Based on the results of the field condition survey and the circular track tests, the following conclusions appear to be justified:

1. There is adequate evidence to show that volcanic cinders having the proper characteristics can be used successfully as base-course material for thin bituminous surface treatments.

2. The investigations reported indicate strongly that volcanic cinders will prove satisfactory for use as base courses for thin bituminous surface treatments if they meet the A. A. S. H. O. specification M 56–38 for type C stabilized base-course materials with the following modifications:

   a. The maximum size may be increased from ¾ inch to 1½ inches.

   b. The amount of material passing the No. 200 sieve should not exceed 20 percent.

   c. If the fraction passing the No. 40 sieve is nonplastic, the maximum permissible liquid limit may be increased from 25 to 30.

   d. The percentage passing the No. 200 sieve may be less than 0.6 of the fraction passing the No. 40 sieve.

3. The base thickness of 5 inches appears to be adequate for the conditions existing on the two projects studied in the field. It should be noted that in most instances where examinations were made, a cinder subbase having a minimum thickness of 8 inches was encountered.

4. Failure occurred in a plastic base course on project 105 C even though this base course rested on a nonplastic cinder subbase.

5. Even though the most satisfactory cinders are nonplastic, adequate compaction and density of base course can be obtained by the judicious use of water during rolling.

6. There was no evidence on the projects studied in the field or in the circular track to indicate that, as a result of manipulation with water, the cinders undergo sufficient change in gradation or physical characteristics to influence their performance as base courses.

7. The one tufa gravel investigated in the circular track did not give satisfactory service as a base course, and this could be explained on the basis of an excess of fine-size material. Tufa gravel with a coarser gradation would probably serve satisfactorily as a base material.

8. This investigation provided a direct correlation between circular track tests and field service behavior. It established the fact that suitable base-course materials will withstand concentrated traffic in the circular track with water 2½ inches above the top of the subbase but that water 4½ inches above the top of the subbase provides a condition more severe than can reasonably be expected under normal service conditions.

(Continued from page 124)

pavement elevation as practicable, preferably within 6 inches. Small pieces of reflectorized material fastened to miscellaneous posts and trees along the roadway are not satisfactory where it is necessary to delineate the alinement of a highway.

TABLE 12.—*Effectiveness of road edge delineators*

Reflecting material	Spacing	Sky condition	Number of observations	Distribution of visibility and effectiveness ratings: Good	Fair	Poor	Rating factor [1]
	Feet			*Percent*	*Percent*	*Percent*	
R. C. type A.[2]	(3)	Dark	52	53	40	7	73
		Moon	24	20	60	20	50
		Total	76	45	45	10	68
Plastic buttons [4]	20	Dark	15	100	0	0	100
		Moon	6	100	0	0	100
		Total	21	100	0	0	100
Do.[4]	40	Dark	37	87	13	0	94
		Moon	18	67	33	0	84
		Total	55	81	19	0	91
Do.[4]	80	Dark	7	71	29	0	86
		Moon	6	75	25	0	88
		Total	13	73	27	0	87
Do.[4]	160	Dark	16	75	25	0	88

[1] Obtained by adding all of the percentage rated "good" to one-half the percentage rated "fair."

[2] Constructed of 1 inch x 3 inch board wrapped with material having a reflective coating for 42 inches and placed 5 feet from pavement edge.

[3] Individual markers.

[4] Delineators constructed of 1 inch x 3 inch black board with three units of reflecting buttons (each unit containing 3 buttons) mounted 1 foot, 2 feet, and 3 feet above and 5 feet outside the pavement edge

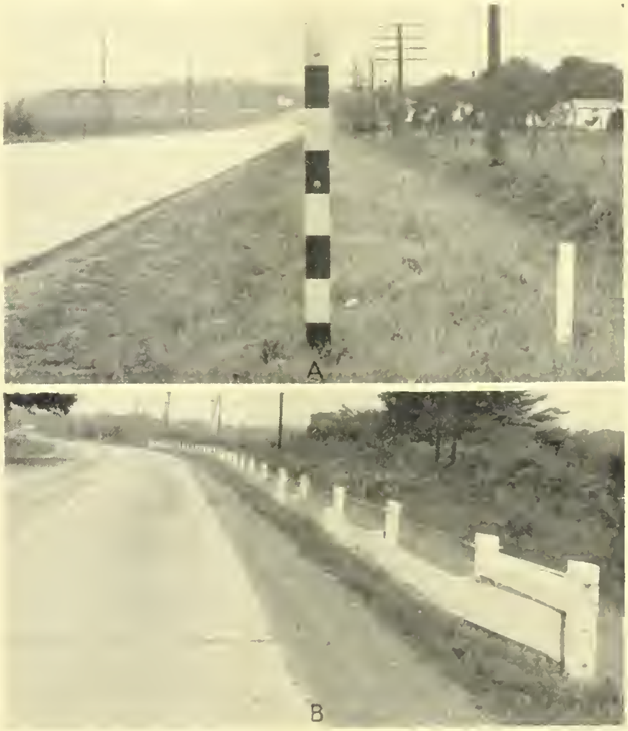

FIGURE 14.—A, 3-INCH BOARD COVERED WITH REFLECTORIZED MATERIAL SUITABLE FOR USE AS A MARKER OR DELINEATOR; B, END PANELS OF GUARDRAIL REFLECTORIZED WITH BEADED WHITE PAINT.

Table 13 shows the rating factors for a number of devices used to mark guardrails and culvert headwalls along the test route. All of the markings were about equally effective, except for the guardrail that was painted white and reflectorized with glass beads which was particularly outstanding (fig. 14–B). For traffic to move with any degree of safety during blackouts, all bridge piers, abutments, and other objects within the roadway or so near as to constitute traffic hazards should be marked either with a cluster of reflecting buttons or a reflectorized sign within 18 inches of the elevation of the roadway surface, or reflectorized with beaded paint or reflectorized materials from a point as close to the pavement surface as practicable to a height of 36 inches, or lanterns suitable for blackout purposes should be used.

CONCLUSIONS

The main conclusions regarding highway signs and markings for blackout conditions based on the results of this study conducted under actual road and operating conditions may be summarized as follows:

1. Both urban and rural highways can be marked so that vehicles equipped with the approved blackout lighting system may be operated with comparative safety at speeds up to at least 20 miles per hour.

2. The need for highway signs, especially of the informational type, is greatly reduced while the need for pavement and object markings is increased.

3. Signs used during normal illumination are of little value. To be effective they must be either illuminated or reflectorized and mounted close to the roadway at

Table 13.—*Visibility or effectiveness of various devices to mark location of guardrails or culverts*

Type of device	Sky condition	Number of observations	Distribution of effectiveness and visibility ratings			Rating factor[1]
			Good	Fair	Poor	
			Percent	*Percent*	*Percent*	
Guardrail painted white	Dark	15	27	46	27	50
	Moon	6	25	50	25	50
	Total	21	26	47	27	50
Reflecting buttons on end post of guardrail.	Dark	45	13	60	27	43
	Moon	18	20	40	40	40
	Total	63	15	55	30	43
Reflecting buttons along steel ribbon of guardrail (1 for each 8-foot section).	Dark	30	46	31	23	62
	Moon	12	40	20	40	50
	Total	42	44	28	28	58
Reflecting buttons on steel ribbon and near top of every fourth post.	Dark	15	40	30	30	55
	Moon	6	25	50	25	50
	Total	21	36	36	28	54
White beaded paint on 3 end panels of guardrail.	Dark	15	70	20	10	80
	Moon	6	100	0	0	100
	Total	21	82	12	6	88
Reflecting material (8 inches x 12 inches) on end post.	Dark	30	21	50	29	46
	Moon	12	60	20	20	70
	Total	42	32	42	26	53
Culvert marker with black and white stripes and 3 reflecting buttons at different heights.	Dark	30	23	77	0	62
	Moon	12	25	50	25	50
	Total	42	24	70	6	59

[1] Obtained by adding all of the percentage rated "good" to one-half the percentage rated "fair."

the proper angle, with the top of their 6-inch legends within 18 inches of the elevation of the road surface.

4. Interior-illuminated signs are the most effective type and must be used whenever a legibility distance in excess of 75 feet is required.

5. Exterior-illuminated signs are most desirable where the same sign must be effective for both normal and blackout conditions. They are satisfactory on 2- and 4-lane highways where a target value in excess of 75 feet is desired and where there is no need of an average legibility distance in excess of 50 feet.

6. Reflectorized signs can be made that will be legible to drivers on both 2- and 4-lane highways, when properly placed, for a distance of at least 50 feet, which is adequate for most locations where a sign is necessary.

7. Reflectorized symbols are effective when properly placed; they are relatively inexpensive and should be used in advance of curves or intersections requiring a reduction in speed below 20 miles per hour.

8. Pavement and object markings to be effective must be reflectorized.

9. Visible centerlines are the most effective and necessary marking. They should be continuous on curves. Dashed centerlines are adequate on tangent sections of highways, providing the dashes are at least 10 feet long and the spaces do not exceed 20 feet.

10. Curbs and objects within the direct line of traffic must be marked.

11. Effort at the present time with respect to highway marking should be directed primarily toward installing markings that are most effective and necessary for both normal and blackout conditions.

INDEX TO PUBLIC ROADS, VOLUME 22, NOW AVAILABLE

The index to PUBLIC ROADS, volume 22, is now available. A chronological list of articles and a list of authors are included with the index. The index will be sent free to subscribers to PUBLIC ROADS requesting it. Requests should be addressed to the Public Roads Administration, Federal Works Agency, Washington, D. C.

Indexes to volumes 6 to 8 and 10 to 21, inclusive, are also available and will be sent to PUBLIC ROADS subscribers upon request. Indexes to volumes 1 to 5, inclusive, have never been prepared. The supply of the index to volume 9 is exhausted.

DISPOSITION OF STATE MOTOR-FUEL TAX RECEIPTS - 1941

COMPILED FOR CALENDAR YEAR FROM REPORTS OF STATE AUTHORITIES

STATE	NET TOTAL RECEIPTS OF CALENDAR YEAR	ADJUSTMENTS DUE TO UNDISTRIBUTED FUNDS, ETC. 1/	NET TOTAL FUNDS DISTRIBUTED 2/	EXPENSES OF COLLECTION AND ADMINISTRATION	FOR OTHER ADMINISTRATIVE PURPOSES 3/	FOR STATE HIGHWAY PURPOSES							FOR LOCAL ROADS AND STREETS				FOR NONHIGHWAY PURPOSES						STATE
						CONSTRUCTION, MAINTENANCE, AND ADMINISTRATION		STATE HIGHWAY POLICE	SERVICE OF STATE HIGHWAY OBLIGATIONS			TOTAL FOR STATE HIGHWAY PURPOSES 5/	FOR WORK ON COUNTY AND LOCAL ROADS 4/ 7/	FOR WORK ON CITY STREETS 8/	SERVICE OF LOCAL HIGHWAY OBLIGATIONS	TOTAL	TO GENERAL FUNDS 9/		FOR RELIEF OF UNEMPLOYMENT OR DESTITUTION	FOR EDUCATION	FOR OTHER PURPOSES 10/	TOTAL	
						ON STATE HIGHWAYS 4/	ON STATE PARK AND FOREST ROADS, ETC. 5/		STATE HIGHWAY BONDS AND NOTES	REIMBURSEMENT OBLIGATIONS 6/	TOTAL						INSPECTION FEES, DEALERS LICENSES, ETC.	MOTOR-FUEL TAX					
	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	
ALABAMA	18,401	-119	18,282	24	78	6,562	39	-	1,949	-	1,949	8,550	9,505	125	-	9,630	-	-	-	-	-	-	ALABAMA
ARIZONA	5,362	-	5,362	69	-	3,541	-	134	-	-	-	3,675	•1,618	-	-	1,618	-	-	-	-	-	-	ARIZONA
ARKANSAS	13,219	-	13,219	341	3	3,824	-	28	7,043	-	7,043	10,895	1,416	97	373	1,886	94	-	-	-	-	94	ARKANSAS
CALIFORNIA	58,094	-1,771	56,323	203	-	33,020	-	-	-	-	-	33,020	•15,927	4,430	-	20,357	-	-	2,743	-	-	2,743	CALIFORNIA
COLORADO	8,833	-263	8,570	86	28	4,342	-	268	1,563	-	1,563	6,173	2,283	-	-	2,283	-	-	-	-	-	-	COLORADO
CONNECTICUT	11,804	-105	11,699	43	-	10,247	-	-	-	1,409	1,409	11,656	-	-	-	-	-	-	-	-	-	-	CONNECTICUT
DELAWARE	2,504	-	2,504	8	-	4/ 868	-	201	91	299	390	1,459	(4/)	-	-	-	-	1,037	-	-	-	1,037	DELAWARE
FLORIDA	30,397	-7	30,390	34	139	17,414	-	1	-	8,132	8,132	25,547	-	-	-	-	385	2,132	-	2,130	23	4,670	FLORIDA
GEORGIA	26,086	-2,396	23,690	782	-	11,575	-	-	77	-	77	11,652	4,971	-	-	4,971	33	1,932	-	4,320	-	6,285	GEORGIA
IDAHO	5,218	45	5,263	18	1	3,712	-	12	112	-	112	3,836	•1,407	1	-	1,408	-	-	-	-	-	-	IDAHO
ILLINOIS	45,261	-31	45,230	197	304	14,425	-	-	-	-	-	14,425	11,200	13,474	905	25,579	194	-	4,531	-	-	4,725	ILLINOIS
INDIANA	29,244	-	29,244	80	110	14,838	-	760	-	-	-	15,598	9,754	2,230	-	11,984	590	882	-	-	-	1,472	INDIANA
IOWA	15,400	-151	15,249	151	-	3,198	-	10	-	3,268	3,268	6,476	•8,616	6	-	8,622	-	-	-	-	-	-	IOWA
KANSAS	11,389	-	11,389	363	118	7,386	-	136	230	686	916	8,438	2,470	-	-	2,470	-	-	-	-	-	-	KANSAS
KENTUCKY	16,180	509	16,689	46	-	14,703	-	253	-	-	-	14,956	1,687	-	-	1,687	-	-	-	-	-	-	KENTUCKY
LOUISIANA	21,562	83	21,645	74	107	6,446	-	-	7,706	-	7,706	14,152	-	-	-	-	-	3,107	1,169	1,518	1,518	7,312	LOUISIANA
MAINE	6,654	-	6,654	17	-	4,157	-	198	1,753	-	1,753	6,108	529	-	-	529	-	-	-	-	-	-	MAINE
MARYLAND	13,070	-	13,070	32	-	7,090	-	-	18	-	18	7,108	1,691	3,821	418	5,930	-	-	-	-	-	-	MARYLAND
MASSACHUSETTS	22,536	143	22,679	50	25	5,196	11/ 1,026	455	12/ 2,558	-	2,558	9,235	11,325	-	12/ 1,387	12,712	24	-	12/ 628	-	5	657	MASSACHUSETTS
MICHIGAN	35,504	-	35,504	255	-	25,688	-	-	3,000	-	3,000	28,688	•6,561	-	-	6,561	-	-	-	-	-	-	MICHIGAN
MINNESOTA	19,055	53	19,108	71	74	12,299	-	314	-	-	-	12,613	•6,307	-	-	6,307	43	-	-	-	-	43	MINNESOTA
MISSISSIPPI	13,753	-2	13,751	123	10	5,680	13/ 483	-	1,765	-	1,765	7,928	•5,689	1	-	5,690	-	-	-	-	-	-	MISSISSIPPI
MISSOURI	14,451	-	14,451	58	49	9,253	-	335	4,572	68	4,640	14,228	-	-	-	-	116	-	-	-	-	116	MISSOURI
MONTANA	5,4[illegible]1	24	5,495	23	4	5,413	-	-	55	-	55	5,468	-	-	-	-	-	-	-	-	-	-	MONTANA
NEBRASKA	12,35[illegible]	39	12,396	123	56	6,227	5	-	-	-	-	6,232	•3,178	352	-	3,530	36	-	2,419	-	-	2,455	NEBRASKA
NEVADA	1,7[illegible]8	-	1,708	11	24	1,670	3	-	-	-	-	1,673	-	-	-	-	-	-	-	-	-	-	NEVADA
NEW HAMPSHIRE	3,716	1	3,717	(14/)	-	2,432	-	-	815	-	815	3,247	238	-	15/ 232	470	-	-	-	-	-	-	NEW HAMPSHIRE
NEW JERSEY	26,306	-222	26,084	201	-	9,003	230	356	2,509	-	2,509	12,098	3,982	-	890	4,872	-	-	7,289	485	1,139	8,913	NEW JERSEY
NEW MEXICO	5,102	-	5,102	145	3	3,012	-	-	1,942	-	1,942	4,954	-	-	-	-	-	-	-	-	-	-	NEW MEXICO
NEW YORK 16/	74,[illegible]0	-5,453	69,067	115	-	6,510	464	627	3,549	-	3,549	11,150	8,729	1,861	-	10,590	-	47,212	-	-	-	47,212	NEW YORK 16/
NORTH CAROLINA	1,9[illegible]1	18	1,949	(17/)	92	4/ 24,329	-	-	5,989	128	6,117	30,446	(4/)	-	-	-	1,248	-	-	-	163	1,411	NORTH CAROLINA
NORTH DAKOTA	3,6[illegible]5	-1	3,624	46	83	2,615	-	-	-	-	-	2,615	874	-	-	874	-	6	-	-	-	6	NORTH DAKOTA
OHIO	18/ 58,11[illegible]	-79	58,034	175	-	24,030	-	-	-	-	-	24,030	10,680	8,010	-	18,690	-	15,139	-	-	-	15,139	OHIO
OKLAHOMA	[illegible]8,976	-57	18,919	459	55	10,697	-	-	-	-	-	10,697	•4,351	763	-	5,114	231	2,333	-	-	-	2,564	OKLAHOMA
OREGON	[illegible]	-95	12,916	49	-	8,453	16	245	1,966	8	1,974	10,688	•2,064	4	-	2,068	-	-	-	-	111	111	OREGON
PENNSYLVANIA	[illegible]	-	[illegible]	[illegible]	-	[illegible]	87	2,264	2,876	-	2,876	38,137	•10,246	-	-	10,246	-	16,216	-	-	56	16,272	PENNSYLVANIA
RHODE ISLAND 19/	[illegible]	-	4,275	1	-	1,326	-	126	141	-	141	1,593	28	-	-	28	-	2,639	-	-	-	2,639	RHODE ISLAND 19/
SOUTH CAROLINA	[illegible]	-	[illegible]	[illegible]	19	10,379	-	-	2,131	482	2,613	12,992	•2,643	95	-	2,738	261	-	-	-	-	261	SOUTH CAROLINA
SOUTH DAKOTA	[illegible]	-12	4,605	43	-	3,834	12	43	16	-	16	3,905	578	-	-	578	79	-	-	-	-	79	SOUTH DAKOTA
TENNESSEE	[illegible]	[illegible]	26,278	410	87	7,226	39	-	4,764	542	5,306	12,571	6,772	-	-	6.772	1,369	-	-	-	5,069	6.438	TENNESSEE
TEXAS	[illegible]	[illegible]	[illegible]	714	-	4,869	-	530	-	9,146	9.146	34,545	•4.387	-	-	4,387	-	17	-	12,937	-	12,954	TEXAS
UTAH	[illegible]	[illegible]	[illegible]	1	13	4,004	-	265	-	-	-	4,269	-	-	-	-	-	-	-	-	-	-	UTAH
VERMONT	[illegible]	-	[illegible]	[illegible]	-	1,592	7	-	21	-	21	1,620	1,012	-	280	1,292	-	-	-	-	15	15	VERMONT
VIRGINIA	[illegible]	[illegible]	22,8[illegible]	[illegible]	[illegible]	4/ 21,5[illegible]	-	478	362	-	362	22,377	4/ 352	64	-	416	-	-	-	-	-	-	VIRGINIA
WASHINGTON	[illegible]	[illegible]	[illegible]	[illegible]	-	[illegible]	-	-	12/ 142	53	195	7,856	•7,367	[illegible],663	12/ 194	10,224	-	-	12/ 1,207	-	-	1,207	WASHINGTON
WEST VIRGINIA	11,5[illegible]9	-	11.559	16	-	4/ 6,801	-	10	4,732	-	4,732	11,543	(4/)	-	-	-	-	-	-	-	-	-	WEST VIRGINIA
WISCONSIN 20/	[illegible]	-	[illegible]	88	189	6,892	165	-	-	2,260	2.260	9,317	5,756	922	-	6,678	-	6,654	-	-	-	6,654	WISCONSIN 20/
WYOMING	[illegible]	-	[illegible]	12	-	2,103	-	60	81	-	81	2,244	758	-	-	758	-	-	-	-	-	-	WYOMING
DISTRICT OF COLUMBIA	[illegible]	1	1,480	(21/)	-	-	-	-	-	-	-	-	-	3,480	-	3,480	-	-	-	-	-	-	DISTRICT OF COLUMBIA
TOTAL	[illegible]	[illegible]	[illegible]	6,104	1,738	460,989	2,576	8,109	54,528	26,481	91,009	562,68[illegible]	176,951	42,399	4,679	[illegible]4,029	4,703	[illegible]	19,986	21,390	8,099	153,484	TOTAL

1/ [illegible] AMOUNTS [illegible] DISTRIBUTED [illegible] IN THE CALENDAR YEAR OFTEN DIFFER FROM ACTUAL COLLECTIONS BECAUSE OF UNDISTRIBUTED FUNDS AND [illegible] BETWEEN ACTIVITIES OF COLLECTING AND EXPENDING AGENCIES.

2/ [illegible] STATES THE PROCEEDS OF HIGHWAY-USER TAXES ARE PLACED IN A COMMON FUND FROM WHICH A DISTRIBUTION IS MADE. THE [illegible] HAVE BEEN PRORATED IN PROPORTION TO THE RECEIPTS NOT OTHERWISE DEDICATED. SEE TABLES MV-3 AND MC-2.

3/ WHERE REPORTED SEPARATELY FROM COLLECTION EXPENSES. FUNDS ALLOTTED FOR MOTOR-FUEL INSPECTION, ADMINISTRATION OF MOTOR [illegible], AND [illegible] OF MOTOR VEHICLES ARE SHOWN IN THIS COLUMN. SEE TABLE SF-9 FOR ADDITIONAL DETAIL OF COLLEC[illegible].

4/ THE FOLLOWING AMOUNTS [illegible] CONSTRUCTION AND MAINTENANCE OF COUNTY ROADS UNDER STATE CONTROL ARE INCLUDED IN ALLOTMENTS [illegible] HIGHWAY PURPOSES: DELAWARE, $[illegible]; NORTH CAROLINA, $[illegible],000; VIRGINIA, $8,579,000; WEST VIRGINIA, $2,309,000. [illegible] CALIFORNIA [illegible] ROADS NOT UNDER STATE CONTROL RECEIVED $35[illegible],000.

5/ [illegible] STATE PARK AND FOREST ROADS, ETC. WERE LISTED SEPARATELY, RATHER THAN UNDER "STATE HIGHWAY [illegible] PRIOR TO 194[illegible].

6/ [illegible] AND LOCAL UNITS OF GOVERNMENT FOR AMOUNTS SPENT ON ROADS NOW ON STATE SYSTEM.

7/ [illegible] LAW PROVIDES THAT THESE FUNDS MAY ALSO BE USED FOR SERVICE OF LOCAL HIGHWAY OBLIGA[illegible] REPORTED SEPARATELY. IN COLORADO FUNDS MAY BE USED ON BOTH STATE AND LOCAL ROADS.

8/ [illegible] ALLOTMENTS FOR CITY STREETS. WHERE REPORTED SEPARATELY, FUNDS ALLOTTED FOR URBAN EXTENSIONS [illegible] ALLOTMENTS FOR STATE HIGHWAY PURPOSES.

9/ [illegible] AS FOLLOWS: LOUISIANA, 1 CENT OF TAX TO PARISHES; WISCONSIN, $[illegible],346,[illegible] PAYMENT TO [illegible] PERSONAL PROPERTY TAXES FORMERLY IMPOSED ON MOTOR VEHICLES. ALLOCATIONS TO COUNTY OR [illegible] IN PART FOR HIGHWAYS, BUT SUCH AMOUNTS WERE NOT REPORTED.

10/ [illegible] FLORIDA, AVIATION; LOUISIANA, HARBOR IMPROVEMENT; MASSACHUSETTS, RIVER AND HARBOR IMPROVE[illegible] LOUISIANA CONSTRUCTION BONDS, $[illegible]43,000, BOARD OF COMMERCE AND NAVIGATION, $376,000, DEFENSE COUNCIL, $500,000, VARIOUS COMMISSIONS, $20,000; NORTH CAROLINA, STATE PROBATION AND PAROLE COMMISSIONS; OREGON, STATE PARKS; PENNSYLVANIA, AVIATION, $51,000, COOPERATIVE WORK OTHER DEPARTMENTS, $[illegible],000; TENNESSEE, DEBT SERVICE ON NONHIGHWAY BONDS; VERMONT, DEBT SERVICE ON NONHIGHWAY PORTION OF FLOOD RELIEF BONDS.

11/ TO METROPOLITAN DISTRICT COMMISSION FOR PARKWAYS AND BOULEVARDS.

12/ INCLUDES DEBT SERVICE CHARGES ON EMERGENCY RELIEF BOND ISSUES, PRORATED IN PROPORTION TO USE OF PROCEEDS FOR STATE HIGHWAYS, LOCAL ROADS AND STREETS, AND NONHIGHWAY PURPOSES.

13/ INCLUDES $287,000 TO GULF COAST COUNTIES FOR SEA-WALL PROTECTION OF STATE HIGHWAYS.

14/ COST OF COLLECTING MOTOR-FUEL TAX WAS NOT REPORTED. EXPENSE OF ADMINISTERING REFUNDS, $4,000, WAS PAID FROM MOTOR-VEHICLE FUNDS.

15/ INCLUDES $116,000 PAID ON HIGHWAY RELIEF BONDS ISSUED BY STATE FOR LOCAL ROADS, AND $116,000 ON PRO-RATA SHARE OF HIGHWAY FLOOD BONDS.

16/ APPROPRIATIONS FOR HIGHWAY PURPOSES OUT OF STATE GENERAL FUND HAVE BEEN CREDITED AGAINST PAYMENTS OF MOTOR-FUEL TAX AND MOTOR-VEHICLE REVENUES TO THE GENERAL FUND AND PRORATED IN PROPORTION TO NET RECEIPTS FROM THESE IMPOSTS NOT OTHERWISE DEDICATED.

17/ INCLUDED WITH MOTOR-VEHICLE COLLECTION EXPENSES.

18/ TAX OF $6[illegible]7,000 ON NON-MOTOR-VEHICLE FUELS NOT INCLUDED.

19/ EXPENDITURES FOR HIGHWAY PURPOSES HAVE BEEN CREDITED AGAINST PAYMENTS OF MOTOR-FUEL TAX AND MOTOR-VEHICLE REVENUES TO THE STATE GENERAL FUND AND PRORATED IN PROPORTION TO NET RECEIPTS FROM THESE IMPOSTS NOT OTHERWISE DEDICATED.

20/ MOTOR-FUEL TAX AND MOTOR-VEHICLE REVENUES ARE PAID INTO HIGHWAY ACCOUNT OF STATE GENERAL FUND, AND ARE USED FOR ANY PURPOSE, INCLUDING HIGHWAYS. EXPENDITURES FOR HIGHWAY PURPOSES HAVE BEEN CREDITED AGAINST RECEIPTS FROM MOTOR-FUEL TAX AND MOTOR-VEHICLE REVENUES, PRORATED IN PROPORTION TO NET RECEIPTS FROM THESE IMPOSTS.

21/ PAID OUT OF GENERAL REVENUE. AMOUNT NOT REPORTED.

DISPOSITION OF STATE MOTOR-VEHICLE RECEIPTS - 1941

COMPILED FOR CALENDAR YEAR FROM REPORTS OF STATE AUTHORITIES

STATE	NET TOTAL RECEIPTS OF CALENDAR YEAR	ADJUSTMENTS DUE TO UNDISTRIBUTED FUNDS, ETC. 1/	NET TOTAL FUNDS DISTRIBUTED 2/	EXPENSES OF COLLECTION AND ADMINISTRATION 3/	FOR OTHER ADMINISTRATIVE PURPOSES 4/	FOR STATE HIGHWAY PURPOSES							FOR LOCAL ROADS AND STREETS				FOR NONHIGHWAY PURPOSES					STATE
						CONSTRUCTION, MAINTENANCE, AND ADMINISTRATION		STATE HIGHWAY POLICE	SERVICE OF STATE HIGHWAY OBLIGATIONS			TOTAL FOR STATE HIGHWAY PURPOSES 6/	FOR WORK ON COUNTY AND LOCAL ROADS 5/ 8/	FOR WORK ON CITY STREETS 9/	SERVICE OF LOCAL HIGHWAY OBLIGATIONS	TOTAL	TO GENERAL FUNDS 10/	FOR RELIEF OF UNEMPLOYMENT OR DESTITUTION	FOR EDUCATION	FOR OTHER PURPOSES 11/	TOTAL	
						ON STATE HIGHWAYS 5/	ON STATE PARK AND FOREST ROADS, ETC. 6/		STATE HIGHWAY BONDS AND NOTES	REIMBURSEMENT OBLIGATIONS 7/	TOTAL											
	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	
ALABAMA	6,431	17	6,448	519	-	2,201	13	669	1,597	-	1,597	4,480	181	266	-	447	1,002	-	-	-	1,002	ALABAMA
ARIZONA	1,352	-29	1,323	291	-	992	-	38	-	-	-	1,030	2	-	-	2	-	-	-	-	-	ARIZONA
ARKANSAS	3,935	283	4,218	92	-	2,107	-	207	1,531	-	1,531	3,845	-	5	223	281	-	-	-	-	-	ARKANSAS
CALIFORNIA	31,927	-1,476	30,451	2,955	35	4,429	-	3,172	3,604	-	3,604	11,205	*4,430	-	-	4,430	11,826	-	-	-	11,826	CALIFORNIA
COLORADO	2,953	-147	2,806	462	-	844	-	52	304	-	304	1,200	1,144	-	-	1,144	-	-	-	-	-	COLORADO
CONNECTICUT	8,249	-192	8,057	992	-	3,511	-	725	-	483	483	4,719	2,346	-	-	2,346	-	-	-	-	-	CONNECTICUT
DELAWARE	1,456	-	1,456	199	-	5/ 437	-	101	46	151	197	735	(5/)	-	-	-	522	-	-	-	522	DELAWARE
FLORIDA	9,463	-	9,463	1,050	-	-	-	559	-	-	-	559	-	-	-	-	15	-	7,839	-	7,854	FLORIDA
GEORGIA	2,951	43	2,994	243	-	1,688	-	799	53	-	53	2,540	114	-	-	114	97	-	-	-	97	GEORGIA
IDAHO	1,542	-108	1,434	82	-	108	-	126	-	-	-	234	1,118	-	-	1,118	-	-	-	-	-	IDAHO
ILLINOIS	26,567	-12	26,555	1,726	-	12,341	-	1,337	9,667	-	9,667	23,345	*1,290	-	-	1,290	194	-	-	-	194	ILLINOIS
INDIANA	11,628	140	11,768	1,170	-	5,525	-	283	-	-	-	5,808	3,632	830	-	4,462	328	-	-	-	328	INDIANA
IOWA	14,231	150	14,381	1,257	-	4,854	-	438	-	4,959	4,959	10,251	2,864	9	-	2,873	-	-	-	-	-	IOWA
KANSAS	4,743	-206	4,537	710	-	2,582	-	62	80	240	320	2,964	863	-	-	863	-	-	-	-	-	KANSAS
KENTUCKY	6,076	-1	6,075	609	-	2,397	-	41	-	-	-	2,438	936	-	-	936	2,092	-	-	-	2,092	KENTUCKY
LOUISIANA	3,486	17	3,503	169	-	2,503	-	630	201	-	201	3,334	-	-	-	-	-	-	-	-	-	LOUISIANA
MAINE	4,434	-14	4,420	214	-	2,634	-	126	1,111	-	1,111	3,871	335	-	-	335	-	-	-	-	-	MAINE
MARYLAND	7,830	-	7,830	363	53	3,191	-	546	699	-	699	4,436	-	1,290	-	1,290	1,688	-	-	-	1,688	MARYLAND
MASSACHUSETTS	7,905	295	8,200	1,581	70	1,507	12/ 297	132	13/ 742	-	742	2,678	3,285	-	13/ 402	3,687	-	13/ 182	-	2	184	MASSACHUSETTS
MICHIGAN	27,163	1,444	28,607	2,141	-	314	-	744	-	-	-	1,058	*25,408	-	-	25,408	-	-	-	-	-	MICHIGAN
MINNESOTA	19,530	-7,851	11,679	840	-	6,622	-	169	2,120	1,762	3,882	10,673	-	-	-	-	166	-	-	-	166	MINNESOTA
MISSISSIPPI	3,433	-36	3,397	208	-	44	-	298	-	-	-	342	*2,847	-	-	2,847	-	-	-	-	-	MISSISSIPPI
MISSOURI	11,713	-27	11,686	653	-	7,174	-	260	3,546	53	3,599	11,033	-	-	-	-	-	-	-	-	-	MISSOURI
MONTANA	1,797	-83	1,714	70	-	-	-	253	-	-	-	253	1,342	49	-	1,391	-	-	-	-	-	MONTANA
NEBRASKA	3,195	11	3,206	272	8	915	-	228	-	-	-	1,143	1,783	-	-	1,783	-	-	-	-	-	NEBRASKA
NEVADA	405	-	405	83	-	239	-	5	78	-	78	322	-	-	-	-	-	-	-	-	-	NEVADA
NEW HAMPSHIRE	3,291	-	3,291	209	4	2,506	-	206	1	-	1	2,713	245	-	14/ 120	365	-	-	-	-	-	NEW HAMPSHIRE
NEW JERSEY	24,349	239	24,588	2,133	-	10,430	266	414	2,906	-	2,906	14,016	4,612	-	1,031	5,643	-	1,493	564	740	2,797	NEW JERSEY
NEW MEXICO	2,213	53	2,266	202	120	736	-	239	-	-	-	975	254	37	-	291	678	-	-	-	678	NEW MEXICO
NEW YORK 15/	53,978	483	54,461	2,822	-	11,197	798	1,078	6,104	-	6,104	19,177	*10,247	5,042	-	15,289	17,173	-	-	-	17,173	NEW YORK 15/
NORTH CAROLINA	9,777	492	10,269	16/ 558	-	5/ 7,093	-	835	1,746	37	1,783	9,711	(5/)	-	-	-	-	-	-	-	-	NORTH CAROLINA
NORTH DAKOTA	2,025	-29	1,996	111	-	919	-	148	-	-	-	1,067	818	-	-	818	-	-	-	-	-	NORTH DAKOTA
OHIO	33,421	-1,834	31,587	3,167	-	8,403	-	873	-	-	-	9,276	*14,102	4,743	-	18,845	-	-	-	299	299	OHIO
OKLAHOMA	7,086	2	7,088	1,023	-	1,399	-	847	-	-	-	2,246	*2,818	786	-	3,604	-	-	215	-	215	OKLAHOMA
OREGON	4,416	-233	4,183	512	-	2,411	5	70	561	2	563	3,049	*589	1	-	590	-	-	-	32	32	OREGON
PENNSYLVANIA	41,326	865	42,191	2,130	-	32,691	87	2,249	2,856	-	2,856	37,883	2,123	-	-	2,123	-	-	-	55	55	PENNSYLVANIA
RHODE ISLAND 17/	3,425	-	3,425	254	-	987	-	93	105	-	105	1,185	21	-	-	21	1,965	-	-	-	1,965	RHODE ISLAND 17/
SOUTH CAROLINA	2,749	-	2,749	240	-	1,502	-	686	235	69	304	2,492	3	14	-	17	-	-	-	-		SOUTH CAROLINA
SOUTH DAKOTA	1,879	31	1,910	79	-	355	1	4	2	-	2	362	1,394	-	-	1,394	75	-	-	-	75	SOUTH DAKOTA
TENNESSEE	7,077	-5	7,072	842	-	4,947	-	402	-	-	-	5,349	-	-	-		881	-	-	-	881	TENNESSEE
TEXAS	26,671	-	26,671	1,918	-	9,688	-	207	-	-	-	9,895	*13,368	-	-	13,368	1,118	-	372	-	1,490	TEXAS
UTAH	1,335	-369	966	136	-	13	-	-	14	-	14	27	463	337	-	800	3	-	-	-	3	UTAH
VERMONT	2,750	1	2,751	84	-	1,389	5	115	19	-	19	1,528	883	-	244	1,127	-	-	-	12	12	VERMONT
VIRGINIA	8,363	60	8,423	16/ 593	-	7,319	-	271	204	-	204	7,794	-	36	-	36	-	-	-	-	-	VIRGINIA
WASHINGTON	5,461	183	5,644	514	-	3,262	2	1,671	-	-	-	4,935	9	-	-	9	-	-	-	186	186	WASHINGTON
WEST VIRGINIA	7,180	1	7,181	155	-	5/ 3,949	-	6	3,071	-	3,071	7,026	(5/)	-	-	-	-	-	-	-	-	WEST VIRGINIA
WISCONSIN 18/	14,862	-	14,862	1,004	-	4,217	101	-	-	1,383	1,383	5,701	3,522	564	-	4,086	4,071	-	-	-	4,071	WISCONSIN 18/
WYOMING	652	10	662	15	-	357	-	10	280	-	280	647	-	-	-	-	-	-	-	-	-	WYOMING
DISTRICT OF COLUMBIA	1,985	-	1,985	199	-	-	-	-	-	-	-	-	-	1,786	-	1,786	-	-	-	-	-	DISTRICT OF COLUMBIA
TOTAL	490,666	-7,832	482,834	37,851	290	184,929	1,575	22,424	43,483	9,139	52,622	261,550	109,391	15,843	2,025	127,259	43,894	1,675	8,989	1,326	55,884	TOTAL

1/ AMOUNTS DISTRIBUTED DURING THE CALENDAR YEAR OFTEN DIFFER FROM ACTUAL COLLECTIONS BECAUSE OF UNDISTRIBUTED FUNDS AND LAG BETWEEN ACCOUNTS OF COLLECTING AND EXPENDING AGENCIES.

2/ IN MANY STATES THE PROCEEDS OF HIGHWAY-USER TAXES ARE PLACED IN A COMMON FUND FROM WHICH A DISTRIBUTION IS MADE. THE AMOUNTS SO DISTRIBUTED HAVE BEEN PRORATED IN PROPORTION TO THE RECEIPTS NOT OTHERWISE DEDICATED. SEE TABLES G-3 AND MC-2.

3/ COLLECTION EXPENSES IN MANY STATES INCLUDE SERVICE CHARGES DEDUCTED BY COUNTY AND LOCAL COLLECTORS. SEE TABLE SF-9 FOR DETAILS OF AMOUNTS INCLUDED IN THIS COLUMN.

4/ WHERE REPORTED SEPARATELY FROM COLLECTION EXPENSES, MISCELLANEOUS EXPENSES OF MOTOR-VEHICLE, MOTOR-FUEL, AND MOTOR-CARRIER REGULATION ARE SHOWN IN THIS COLUMN.

5/ THE FOLLOWING AMOUNTS FOR CONSTRUCTION AND MAINTENANCE OF COUNTY ROADS UNDER STATE CONTROL ARE INCLUDED IN ALLOTMENTS FOR STATE HIGHWAY PURPOSES: DELAWARE, $162,000; NORTH CAROLINA, $2,399,000; WEST VIRGINIA, $1,034,000.

6/ AMOUNTS DISTRIBUTED FOR STATE PARK AND FOREST ROADS, ETC., WERE REPORTED SEPARATELY, RATHER THAN UNDER "STATE HIGHWAY PURPOSES" ON TABLE MV-3 FOR YEARS PRIOR TO 1940.

7/ REIMBURSEMENT TO COUNTIES AND LOCAL UNITS OF GOVERNMENT FOR AMOUNTS SPENT ON ROADS NOW ON STATE SYSTEM.

8/ IN STATES INDICATED BY STAR (*) LAW PROVIDES THAT THESE FUNDS MAY ALSO BE USED FOR SERVICE OF LOCAL HIGHWAY OBLIGATIONS. AMOUNTS SO USED NOT REPORTED SEPARATELY. IN COLORADO FUNDS MAY BE USED ON BOTH STATE AND LOCAL ROADS.

9/ THIS COLUMN SHOWS SPECIFIC ALLOTMENTS FOR CITY STREETS. WHERE REPORTED SEPARATELY, FUNDS ALLOTTED FOR URBAN EXTENSIONS OF STATE HIGHWAY SYSTEM ARE INCLUDED IN ALLOTMENTS FOR STATE HIGHWAY PURPOSES.

10/ TO STATE GENERAL FUNDS EXCEPT IN THE FOLLOWING STATES: ALABAMA, COUNTY AND MUNICIPAL GENERAL FUNDS; CALIFORNIA, COUNTY AND MUNICIPAL GENERAL FUNDS, $9,461,000; NEW MEXICO, COUNTY GENERAL FUNDS, $424,000; WISCONSIN, $1,436,000 TO TOWNS, CITIES, AND VILLAGES IN LIEU OF PERSONAL PROPERTY TAX FORMERLY IMPOSED ON MOTOR VEHICLES. ALLOCATIONS TO COUNTY OR LOCAL GENERAL FUNDS MAY HAVE BEEN USED IN PART FOR HIGHWAYS, BUT SUCH AMOUNTS WERE NOT REPORTED.

11/ FOR THE FOLLOWING PURPOSES: MASSACHUSETTS, RIVER AND HARBOR IMPROVEMENT; NEW JERSEY, DEBT SERVICE ON INSTITUTIONAL CONSTRUCTION BONDS, $281,000, BOARD OF COMMERCE AND NAVIGATION, $436,000, VARIOUS COMMISSIONS, $23,000; OHIO, CARE OF INDIGENT PERSONS INJURED IN MOTOR-VEHICLE ACCIDENTS; OREGON, STATE PARKS; PENNSYLVANIA, AVIATION, $50,000, COOPERATIVE WORK OTHER DEPARTMENTS, $5,000; VERMONT, DEBT SERVICE ON NONHIGHWAY PORTION OF FLOOD RELIEF BONDS.

12/ TO METROPOLITAN DISTRICT COMMISSION FOR PARKWAYS AND BOULEVARDS.

13/ INCLUDES DEBT SERVICE CHARGES ON EMERGENCY RELIEF BOND ISSUES, PRORATED IN PROPORTION TO USE OF PROCEEDS FOR STATE HIGHWAYS, LOCAL ROADS AND STREETS, AND NONHIGHWAY PURPOSES.

14/ DEBT SERVICE ON HIGHWAY RELIEF BONDS, A STATE OBLIGATION INCURRED FOR IMPROVEMENT OF LOCAL ROADS.

15/ APPROPRIATIONS FOR HIGHWAY PURPOSES OUT OF STATE GENERAL FUND HAVE BEEN CREDITED AGAINST PAYMENTS OF MOTOR-FUEL TAX AND MOTOR-VEHICLE REVENUES TO THE GENERAL FUND AND PRORATED IN PROPORTION TO NET RECEIPTS FROM THESE IMPOSTS NOT OTHERWISE DEDICATED.

16/ INCLUDES EXPENSES OF COLLECTING MOTOR-FUEL TAX.

17/ EXPENDITURES FOR HIGHWAY PURPOSES HAVE BEEN CREDITED AGAINST PAYMENTS OF MOTOR-FUEL TAX AND MOTOR-VEHICLE REVENUES TO THE STATE GENERAL FUND AND PRORATED IN PROPORTION TO NET RECEIPTS FROM THESE IMPOSTS NOT OTHERWISE DEDICATED.

18/ MOTOR-FUEL TAX AND MOTOR-VEHICLE REVENUES ARE PAID INTO HIGHWAY ACCOUNT OF STATE GENERAL FUND, AND ARE USED FOR ANY PURPOSE, INCLUDING HIGHWAYS. EXPENDITURES FOR HIGHWAY PURPOSES HAVE BEEN CREDITED AGAINST RECEIPTS FROM MOTOR-FUEL TAX AND MOTOR-VEHICLE REVENUES, PRORATED IN PROPORTION TO NET RECEIPTS FROM THESE IMPOSTS.

DISPOSITION OF STATE MOTOR-CARRIER TAX RECEIPTS - 1941

COMPILED FOR CALENDAR YEAR FROM REPORTS OF STATE AUTHORITIES

STATE	NET TOTAL RECEIPTS OF CALENDAR YEAR	ADJUSTMENTS DUE TO UNDISTRIBUTED FUNDS, ETC. 1/	NET TOTAL FUNDS DISTRIBUTED 2/	EXPENSES OF COLLECTION AND ADMINISTRATION	FOR STATE HIGHWAY PURPOSES: CONSTRUCTION, MAINTENANCE, AND ADMINISTRATION: ON STATE HIGHWAYS 3/	FOR STATE HIGHWAY PURPOSES: CONSTRUCTION, MAINTENANCE, AND ADMINISTRATION: ON STATE PARK AND FOREST ROADS, ETC. 4/	FOR STATE HIGHWAY PURPOSES: STATE HIGHWAY POLICE	FOR STATE HIGHWAY PURPOSES: SERVICE OF STATE HIGHWAY OBLIGATIONS: STATE HIGHWAY BONDS AND NOTES	FOR STATE HIGHWAY PURPOSES: SERVICE OF STATE HIGHWAY OBLIGATIONS: REIMBURSEMENT OBLIGATIONS 5/	FOR STATE HIGHWAY PURPOSES: SERVICE OF STATE HIGHWAY OBLIGATIONS: TOTAL	FOR STATE HIGHWAY PURPOSES: TOTAL FOR STATE HIGHWAY PURPOSES 4/	FOR LOCAL ROADS AND STREETS: FOR WORK ON COUNTY AND LOCAL ROADS 3/ 6/	FOR LOCAL ROADS AND STREETS: FOR WORK ON CITY STREETS 7/	FOR LOCAL ROADS AND STREETS: SERVICE OF LOCAL HIGHWAY OBLIGATIONS	FOR LOCAL ROADS AND STREETS: TOTAL	FOR NONHIGHWAY PURPOSES: TO STATE GENERAL FUNDS	FOR NONHIGHWAY PURPOSES: FOR EDUCATION	FOR NONHIGHWAY PURPOSES: FOR STATE PARKS	FOR NONHIGHWAY PURPOSES: TOTAL	STATE
	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	
ALABAMA	422	3	425	67	329	2	-	-	-	-	331	27	-	-	27	-	-	-	-	ALABAMA
ARIZONA	225	-21	204	42	155	-	6	-	-	-	161	1	-	-	1	-	-	-	-	ARIZONA
ARKANSAS	4	-1	3	3	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	ARKANSAS
CALIFORNIA	3,888	-82	3,806	517	133	-	-	-	-	-	133	1	-	-	1	3,155	-	-	3,155	CALIFORNIA
COLORADO	859	-18	841	101	281	-	17	101	-	101	399	341	-	-	341	-	-	-	-	COLORADO
CONNECTICUT	337	-8	329	74	126	-	-	-	17	17	143	-	112	-	112	-	-	-	-	CONNECTICUT
DELAWARE	(8/)	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	DELAWARE
FLORIDA	427	-	427	90	-	-	-	-	-	-	-	-	11	315	326	3	8	-	11	FLORIDA
GEORGIA	110	1	111	83	-	-	-	-	-	-	-	-	-	-	-	28	-	-	28	GEORGIA
IDAHO	73	-24	49	20	-	-	29	-	-	-	29	-	-	-	-	-	-	-	-	IDAHO
ILLINOIS	(8/)	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	ILLINOIS
INDIANA	1,412	-9	1,403	114	672	-	34	-	-	-	706	442	101	-	543	40	-	-	40	INDIANA
IOWA	415	-8	407	72	62	-	-	-	63	63	125	*210	-	-	210	-	-	-	-	IOWA
KANSAS	1,629	-	1,629	448	800	-	15	25	74	99	914	267	-	-	267	-	-	-	-	KANSAS
KENTUCKY	441	1	442	63	335	-	6	-	-	-	341	38	-	-	38	-	-	-	-	KENTUCKY
LOUISIANA	149	-	149	30	-	-	-	-	-	-	-	-	-	-	-	119	-	-	119	LOUISIANA
MAINE	22	-	22	22	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	MAINE
MARYLAND	(9/)	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	MARYLAND
MASSACHUSETTS	138	-	138	45	-	-	-	-	-	-	-	-	-	-	-	93	-	-	93	MASSACHUSETTS
MICHIGAN	624	-13	611	247	364	-	-	-	-	-	364	-	-	-	-	-	-	-	-	MICHIGAN
MINNESOTA	47	-	47	47	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	MINNESOTA
MISSISSIPPI	239	-	239	50	-	-	-	-	-	-	-	*189	-	-	189	-	-	-	-	MISSISSIPPI
MISSOURI	942	615	1,557	90	733	-	26	362	6	368	1,127	340	-	-	340	-	-	-	-	MISSOURI
MONTANA	66	-25	41	41	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	MONTANA
NEBRASKA	32	-	32	32	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	NEBRASKA
NEVADA	317	-1	316	45	265	-	6	-	-	-	271	-	-	-	-	-	-	-	-	NEVADA
NEW HAMPSHIRE	4	-	4	4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	NEW HAMPSHIRE
NEW JERSEY	104	-	104	-	104	-	-	-	-	-	104	-	-	-	-	-	-	-	-	NEW JERSEY
NEW MEXICO	272	-95	177	73	104	-	-	-	-	-	104	-	-	-	-	-	-	-	-	NEW MEXICO
NEW YORK	26	-	26	26	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	NEW YORK
NORTH CAROLINA	584	40	624	-	3/ 498	-	-	123	3	126	624	(3/)	-	-	-	-	-	-	-	NORTH CAROLINA
NORTH DAKOTA	46	-	46	42	4	-	-	-	-	-	4	-	-	-	-	-	-	-	-	NORTH DAKOTA
OHIO	685	-	685	107	421	-	-	-	-	-	421	118	39	-	157	-	-	-	-	OHIO
OKLAHOMA	29	19	48	14	27	-	-	-	-	-	27	7	-	-	7	-	-	-	-	OKLAHOMA
OREGON	1,664	-2	1,662	235	915	2	61	212	1	213	1,191	*223	1	-	224	-	-	12	12	OREGON
PENNSYLVANIA	7	-	7	-	7	-	-	-	-	-	7	-	-	-	-	-	-	-	-	PENNSYLVANIA
RHODE ISLAND	13	1	14	12	2	-	-	-	-	-	2	-	-	-	-	-	-	-	-	RHODE ISLAND
SOUTH CAROLINA	299	95	394	77	183	-	-	-	-	-	183	-	29	-	29	105	-	-	105	SOUTH CAROLINA
SOUTH DAKOTA	469	75	544	30	485	2	5	2	-	2	494	2	-	-	2	18	-	-	18	SOUTH DAKOTA
TENNESSEE	121	-	121	61	-	-	-	-	-	-	-	-	-	-	-	60	-	-	60	TENNESSEE
TEXAS	177	-	177	169	8	-	-	-	-	-	8	-	-	-	-	-	-	-	-	TEXAS
UTAH 10/	-	19	19	11	1	-	-	-	-	-	1	-	-	-	-	7	-	-	7	UTAH 10/
VERMONT	(8/)	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	VERMONT
VIRGINIA	491	-1	490	84	379	-	14	11	-	11	404	-	2	-	2	-	-	-	-	VIRGINIA
WASHINGTON	192	-	192	192	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	WASHINGTON
WEST VIRGINIA	140	-	140	-	68	-	-	72	-	72	140	-	-	-	-	-	-	-	-	WEST VIRGINIA
WISCONSIN	1,841	-	1,841	86	-	-	-	-	-	-	-	-	-	-	-	1,755	-	-	1,755	WISCONSIN
WYOMING	329	-1	328	30	290	-	8	-	-	-	298	-	-	-	-	-	-	-	-	WYOMING
DISTRICT OF COLUMBIA	268	-	268	-	-	-	-	-	-	-	-	-	-	-	-	268	-	-	268	DISTRICT OF COLUMBIA
TOTAL	20,576	563	21,139	3,596	7,751	6	227	908	164	1,072	9,056	2,206	295	315	2,816	5,651	8	12	5,671	TOTAL

1/ AMOUNTS DISTRIBUTED DURING THE CALENDAR YEAR OFTEN DIFFER FROM ACTUAL COLLECTIONS BECAUSE OF UNDISTRIBUTED FUNDS AND LAG BETWEEN ACCOUNTS OF COLLECTING AND EXPENDING AGENCIES.

2/ IN MANY STATES THE PROCEEDS OF HIGHWAY-USER TAXES ARE PLACED IN A COMMON FUND FROM WHICH A DISTRIBUTION IS MADE. THE AMOUNTS SO DISTRIBUTED HAVE BEEN PRORATED IN PROPORTION TO THE RECEIPTS NOT OVERWISE [illegible]. SEE TABLES [illegible]-3 AND MV-[illegible].

3/ APPROXIMATELY $16[illegible],000 FOR USE ON COUNTY ROADS UNDER STATE CONTROL IN NORTH CAROLINA INCLUDED IN ALLOTMENTS FOR STATE HIGHWAY PURPOSES.

4/ AMOUNTS ALLOTTED FOR STATE PARK AND FOREST ROADS, ETC. WERE REPORTED SEPARATELY, RATHER THAN UNDER "STATE HIGHWAY PURPOSES", ON TABLE MC-2 FOR YEARS PRIOR TO 1940.

5/ REIMBURSEMENT TO COUNTIES AND LOCAL UNITS OF GOVERNMENT FOR AMOUNTS SPENT ON ROADS NOW ON STATE SYSTEM.

6/ IN STATES INDICATED BY STAR (•) LAW PROVIDES THAT THESE FUNDS MAY ALSO BE USED FOR SERVICE OF LOCAL HIGHWAY OBLIGATIONS. AMOUNTS SO USED NOT REPORTED SEPARATELY. IN COLORADO FUNDS MAY BE USED ON BOTH STATE AND LOCAL ROADS.

7/ THIS COLUMN SHOWS SPECIFIC ALLOTMENTS FOR CITY STREETS. WHERE REPORTED SEPARATELY, FUNDS ALLOTTED FOR URBAN EXTENSIONS OF STATE HIGHWAY SYSTEM ARE INCLUDED IN ALLOTMENTS FOR STATE HIGHWAY PURPOSES.

8/ NO SPECIAL TAXES ON MOTOR-CARRIERS REPORTED.

9/ WEIGHT AND PASSENGER-MILE TAXES PAID BY MOTOR CARRIERS IN LIEU OF REGISTRATION FEES ARE INCLUDED IN MOTOR-VEHICLE RECEIPTS, TABLE MV-[illegible].

10/ MOTOR-CARRIER TAXES NO LONGER IMPOSED. AMOUNTS SHOWN REPRESENT DISTRIBUTION OF REVENUES COLLECTED IN PRIOR YEARS.

DISPOSITION OF RECEIPTS FROM STATE IMPOSTS ON HIGHWAY USERS - 1941

COMPILED FOR CALENDAR YEAR FROM REPORTS OF STATE AUTHORITIES

STATE	NET TOTAL RECEIPTS OF CALENDAR YEAR 1/	ADJUSTMENTS DUE TO UNDISTRIBUTED FUNDS, ETC. 2/	NET TOTAL FUNDS DISTRIBUTED	EXPENSES OF COLLECTION AND ADMINISTRATION 3/	FOR STATE HIGHWAY PURPOSES							FOR LOCAL ROADS AND STREETS				FOR NONHIGHWAY PURPOSES						STATE
					CONSTRUCTION, MAINTENANCE, AND ADMINISTRATION		STATE HIGHWAY POLICE	SERVICE OF STATE HIGHWAY OBLIGATIONS			TOTAL FOR STATE HIGHWAY PURPOSES 2/	FOR WORK ON COUNTY AND LOCAL ROADS 4/ 7/	FOR WORK ON CITY STREETS 8/	SERVICE OF LOCAL HIGHWAY OBLIGATIONS	TOTAL	TO GENERAL FUNDS 9/		FOR RELIEF OF UNEMPLOYMENT OR DESTITUTION	FOR EDUCATION	FOR OTHER PURPOSES 10/	TOTAL	
					ON STATE HIGHWAYS 4/	ON STATE PARK AND FOREST ROADS, ETC. 5/		STATE HIGHWAY BONDS AND NOTES	REIMBURSEMENT OBLIGATIONS 6/	TOTAL						MOTOR-FUEL INSPECTION FEES, DEALERS LICENSES, ETC.	ALL OTHER HIGHWAY-USER IMPOSTS					
	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	1,000 DOLLARS	
ALABAMA	25,254	-99	25,155	688	9,092	54	669	3,546	-	3,546	13,361	9,713	391	-	10,104	-	1,002	-	-	-	1,002	ALABAMA
ARIZONA	6,939	-50	6,889	402	4,688	-	178	-	-	-	4,866	•1,621	-	-	1,621	-	-	-	-	-	-	ARIZONA
ARKANSAS	17,158	282	17,440	439	5,931	-	235	8,574	-	8,574	14,740	1,416	150	601	2,167	94	-	-	-	-	94	ARKANSAS
CALIFORNIA	93,909	-3,329	90,580	3,710	37,582	-	3,172	3,604	-	3,604	44,358	•20,358	4,430	-	24,788	-	14,981	2,743	-	-	17,724	CALIFORNIA
COLORADO	12,645	-428	12,217	677	5,467	-	337	1,968	-	1,968	7,772	3,768	-	-	3,768	-	-	-	-	-	-	COLORADO
CONNECTICUT	20,390	-305	20,085	1,109	13,884	-	725	-	1,909	1,909	16,518	2,346	112	-	2,458	-	-	-	-	-	-	CONNECTICUT
DELAWARE	3,960	-	3,960	207	4/ 1,305	-	302	137	450	587	2,194	(4/)	-	-	-	-	1,559	-	-	-	1,559	DELAWARE
FLORIDA	40,287	-7	40,280	1,313	17,414	-	560	-	8,132	8,132	26,106	-	11	315	326	385	2,150	-	9,977	23	12,535	FLORIDA
GEORGIA	29,147	-2,352	26,795	1,108	13,263	-	799	130	-	130	14,192	5,085	-	-	5,085	33	2,057	-	4,320	-	6,410	GEORGIA
IDAHO	6,833	-87	6,746	121	3,820	-	167	112	-	112	4,099	•2,525	1	-	2,526	-	-	-	-	-	-	IDAHO
ILLINOIS	71,828	-43	71,785	2,227	26,766	-	1,337	9,667	-	9,667	37,770	12,490	13,474	905	26,869	194	194	4,531	-	-	4,919	ILLINOIS
INDIANA	42,284	131	42,415	1,474	21,035	-	1,077	-	-	-	22,112	13,828	3,161	-	16,989	590	1,250	-	-	-	1,840	INDIANA
IOWA	30,046	-9	30,037	1,480	8,114	-	448	-	8,290	8,290	16,852	•11,690	15	-	11,705	-	-	-	-	-	-	IOWA
KANSAS	17,761	-206	17,555	1,639	10,768	-	213	335	1,000	1,335	12,316	3,600	-	-	3,600	-	-	-	-	-	-	KANSAS
KENTUCKY	22,697	509	23,206	718	17,435	-	300	-	-	-	17,735	2,661	-	-	2,661	-	2,092	-	-	-	2,092	KENTUCKY
LOUISIANA	25,197	100	25,297	380	8,949	-	630	7,907	-	7,907	17,486	-	-	-	-	-	3,226	1,169	1,518	1,518	7,431	LOUISIANA
MAINE	11,110	-14	11,096	253	6,791	-	324	2,864	-	2,864	9,979	864	-	-	864	-	-	-	-	-	-	MAINE
MARYLAND	20,900	-	20,900	448	10,281	-	546	717	-	717	11,544	1,691	5,111	418	7,220	-	1,688	-	-	-	1,688	MARYLAND
MASSACHUSETTS	30,579	438	31,017	1,771	6,703	11/ 1,323	587	12/ 3,300	-	3,300	11,913	14,610	-	12/ 1,789	16,399	24	93	12/ 810	-	7	934	MASSACHUSETTS
MICHIGAN	63,291	1,431	64,722	2,643	26,366	-	744	3,000	-	3,000	30,110	•31,969	-	-	31,969	-	-	-	-	-	-	MICHIGAN
MINNESOTA	38,632	-7,798	30,834	1,032	18,921	-	483	2,120	1,762	3,882	23,286	•6,307	-	-	6,307	43	166	-	-	-	209	MINNESOTA
MISSISSIPPI	17,425	-38	17,387	391	5,724	13/ 483	298	1,765	-	1,765	8,270	•8,725	1	-	8,726	-	-	-	-	-	-	MISSISSIPPI
MISSOURI	27,106	588	27,694	850	17,160	-	621	8,480	127	8,607	26,388	340	-	-	340	116	-	-	-	-	116	MISSOURI
MONTANA	7,334	-84	7,250	138	5,413	-	253	55	-	55	5,721	1,342	49	-	1,391	-	-	-	-	-	-	MONTANA
NEBRASKA	15,584	50	15,634	491	7,142	5	228	-	-	-	7,375	•4,961	352	-	5,313	36	-	2,419	-	-	2,455	NEBRASKA
NEVADA	2,430	-1	2,429	163	2,174	3	11	78	-	78	2,266	-	-	-	-	-	-	-	-	-	-	NEVADA
NEW HAMPSHIRE	7,011	1	7,012	217	4,938	-	206	816	-	816	5,960	483	-	14/ 352	835	-	-	-	-	-	-	NEW HAMPSHIRE
NEW JERSEY	50,759	17	50,776	2,334	19,537	496	770	5,415	-	5,415	26,218	8,594	-	1,921	10,515	-	-	8,782	1,048	1,879	11,709	NEW JERSEY
NEW MEXICO	7,587	-42	7,545	543	3,852	-	239	1,942	-	1,942	6,033	254	37	-	291	-	678	-	-	-	678	NEW MEXICO
NEW YORK 15/	128,524	-4,970	123,554	2,963	17,707	1,262	1,705	9,653	-	9,653	30,327	•18,976	6,903	-	25,879	-	64,385	-	-	-	64,385	NEW YORK 15/
NORTH CAROLINA	42,292	550	42,842	650	4/ 31,920	-	835	7,858	168	8,026	40,781	(4/)	-	-	-	1,248	-	-	-	163	1,411	NORTH CAROLINA
NORTH DAKOTA	5,696	-30	5,666	282	3,538	-	148	-	-	-	3,686	1,692	-	-	1,692	-	6	-	-	-	6	NORTH DAKOTA
OHIO	92,219	-1,913	90,306	3,449	32,854	-	873	-	-	-	33,727	•24,900	12,792	-	37,692	-	15,139	-	-	299	15,438	OHIO
OKLAHOMA	26,091	-36	26.055	1,581	12,123	-	847	-	-	-	12,970	•7,176	1,549	-	8,725	231	3,333	-	215	-	3,779	OKLAHOMA
OREGON	19,091	-330	18,761	796	11,779	23	376	2,739	11	2,750	14,928	•2,876	6	-	2,882	-	-	-	-	155	155	OREGON
PENNSYLVANIA	106,205	865	107,070	2,347	65,608	174	4,513	5,732	-	5,732	76,027	•12,369	-	-	12,369	-	16,216	-	-	111	16,327	PENNSYLVANIA
RHODE ISLAND 16/	7,713	1	7,714	281	2,315	-	219	246	-	246	2,780	49	-	-	49	-	4,604	-	-	-	4,604	RHODE ISLAND 16/
SOUTH CAROLINA	19,105	98	19,203	386	12,064	-	686	2,366	551	2,917	15,667	•2,646	138	-	2,784	261	105	-	-	-	366	SOUTH CAROLINA
SOUTH DAKOTA	6,965	94	7,059	152	4,674	15	52	20	-	20	4,761	1,974	-	-	1,974	79	93	-	-	-	172	SOUTH DAKOTA
TENNESSEE	32,597	874	33,471	1,400	12,173	39	402	4,764	542	5,306	17,920	6,772	-	-	6,772	1,369	941	-	-	5,069	7,379	TENNESSEE
TEXAS	79,616	-168	79,448	2,801	34,565	-	737	-	9,146	9,146	44,448	•17,755	-	-	17,755	-	1,135	-	13,309	-	14,444	TEXAS
UTAH	5,705	-421	5,284	177	4,018	-	265	14	-	14	4,297	463	337	-	800	-	10	-	-	-	10	UTAH
VERMONT	5,680	1	5,681	87	2,981	12	115	40	-	40	3,148	1,895	-	524	2,419	-	-	-	-	27	27	VERMONT
VIRGINIA	31,750	-7	31,743	714	4/ 29,235	-	763	577	-	577	30,575	4/ 352	102	-	454	-	-	-	-	-	-	VIRGINIA
WASHINGTON	24,987	183	25,170	753	10,923	2	1,671	12/ 142	53	195	12,791	•7,376	2,663	12/ 194	10,233	-	-	12/ 1,207	-	186	1,393	WASHINGTON
WEST VIRGINIA	18,879	1	18,880	171	4/ 10,818	-	16	7,875	-	7,875	18,709	(4/)	-	-	-	-	-	-	-	-	-	WEST VIRGINIA
WISCONSIN 17/	39,629	-	39,629	1,367	11,109	266	-	-	3,643	3,643	15,018	9,278	1,486	-	10,764	-	12,480	-	-	-	12,480	WISCONSIN 17/
WYOMING	3,995	9	4,004	57	2,750	-	78	361	-	361	3,189	758	-	-	758	-	-	-	-	-	-	WYOMING
DISTRICT OF COLUMBIA	5,732	1	5,733	199	-	-	-	-	-	-	-	-	5,266	-	5,266	-	268	-	-	-	268	DISTRICT OF COLUMBIA
TOTAL	1,468,554	-16,543	1,452,011	49,579	653,669	4,157	30,760	108,919	35,784	144,703	833,289	288,548	58,537	7,019	354,104	4,703	148,851	21,661	30,387	9,437	215,039	TOTAL

1/ INCLUDES RECEIPTS FROM MOTOR-FUEL TAXES, MOTOR-VEHICLE FEES AND FINES, AND SPECIAL IMPOSTS ON MOTOR VEHICLES OPERATED FOR HIRE (MOTOR-CARRIER TAXES). SEE TABLES G-3, MV-3, AND MC-2, WHICH GIVE DISTRIBUTION OF RECEIPTS SEPARATELY.

2/ AMOUNTS DISTRIBUTED DURING THE CALENDAR YEAR OFTEN DIFFER FROM ACTUAL COLLECTIONS BECAUSE OF UNDISTRIBUTED FUNDS AND LAG BETWEEN ACCOUNTS OF COLLECTING AND EXPENDING AGENCIES.

3/ INCLUDES EXPENSES OF COLLECTION AND ADMINISTRATION OF MOTOR-FUEL TAX, MOTOR-VEHICLE FEES, AND MOTOR-CARRIER TAXES, AND MISCELLANEOUS EXPENSES OF MOTOR-VEHICLE REGULATION.

4/ THE FOLLOWING AMOUNTS FOR CONSTRUCTION AND MAINTENANCE OF COUNTY ROADS UNDER STATE CONTROL ARE INCLUDED IN ALLOTMENTS FOR STATE HIGHWAY PURPOSES: DELAWARE, $484,000; NORTH CAROLINA, $10,796,000; VIRGINIA, $8,579,000; WEST VIRGINIA, $[illegible],343,000.

5/ AMOUNTS DISTRIBUTED FOR STATE PARK AND FOREST ROADS, ETC. WERE REPORTED SEPARATELY, RATHER THAN UNDER "STATE HIGHWAY PURPOSES", ON TABLE OF FOR YEARS PRIOR TO 1940.

6/ REIMBURSEMENT TO COUNTIES AND LOCAL UNITS OF GOVERNMENT FOR AMOUNTS SPENT ON ROADS NOW ON STATE SYSTEM.

7/ IN STATES INDICATED BY STAR (•) LAW PROVIDES THAT THESE FUNDS MAY ALSO BE USED FOR SERVICE OF LOCAL HIGHWAY OBLIGATIONS, AMOUNTS SO USED NOT REPORTED SEPARATELY. IN COLORADO FUNDS MAY BE USED ON BOTH STATE AND LOCAL ROADS.

8/ THIS COLUMN SHOWS SPECIFIC ALLOTMENTS FOR CITY STREETS. WHERE REPORTED SEPARATELY, FUNDS ALLOTTED FOR URBAN EXTENSIONS OF STATE HIGHWAY SYSTEM ARE INCLUDED IN ALLOTMENTS FOR STATE HIGHWAY PURPOSES.

9/ TO STATE GENERAL FUNDS EXCEPT AS FOLLOWS: ALABAMA, COUNTY AND MUNICIPAL GENERAL FUNDS; CALIFORNIA, $9,461,000 TO COUNTY AND MUNICIPAL GENERAL FUNDS; LOUISIANA, PARISH GENERAL FUNDS; NEW MEXICO, $424,000 TO COUNTY GENERAL FUNDS; WISCONSIN, $1,436,000 TO TOWNS, CITIES, AND VILLAGES IN LIEU OF PERSONAL PROPERTY TAX FORMERLY IMPOSED ON MOTOR VEHICLES. ALLOCATIONS TO COUNTY OR LOCAL GENERAL FUNDS MAY HAVE BEEN USED IN PART FOR HIGHWAYS, BUT SUCH AMOUNTS WERE NOT REPORTED.

10/ FOR THE FOLLOWING PURPOSES: FLORIDA, AVIATION; LOUISIANA, HARBOR IMPROVEMENT; MASSACHUSETTS, RIVER AND HARBOR IMPROVEMENT; NEW JERSEY, DEBT SERVICE ON INSTITUTIONAL CONSTRUCTION BONDS, $524,000; BOARD OF COMMERCE AND NAVIGATION, $812,000, DEFENSE COUNCIL, $500,000, VARIOUS COMMISSIONS, $43,000; NORTH CAROLINA, STATE PROBATION AND PAROLE COMMISSIONS; OHIO, CARE OF INDIGENT PERSONS INJURED IN MOTOR-VEHICLE ACCIDENTS; OREGON, STATE PARKS; PENNSYLVANIA, AVIATION, $101,000, COOPERATIVE WORK OTHER DEPARTMENTS, $10,000; TENNESSEE, DEBT SERVICE ON NONHIGHWAY BONDS; VERMONT, DEBT SERVICE ON NONHIGHWAY PORTION OF FLOOD RELIEF BONDS.

11/ TO METROPOLITAN DISTRICT COMMISSION FOR PARKWAYS AND BOULEVARDS.

12/ INCLUDES DEBT SERVICE CHARGES ON EMERGENCY RELIEF BOND ISSUES, PRORATED IN PROPORTION TO USE OF PROCEEDS FOR STATE HIGHWAYS, LOCAL ROADS AND STREETS, AND NONHIGHWAY PURPOSES.

13/ INCLUDES $287,000 TO GULF COAST COUNTIES FOR SEA-WALL PROTECTION OF STATE HIGHWAYS.

14/ INCLUDES $236,000 PAID ON HIGHWAY RELIEF BONDS ISSUED BY STATE FOR LOCAL ROADS, AND $116,000 ON PRO-RATA SHARE OF HIGHWAY FLOOD BONDS.

15/ APPROPRIATIONS FOR HIGHWAY PURPOSES OUT OF STATE GENERAL FUND HAVE BEEN CREDITED AGAINST PAYMENTS OF MOTOR-FUEL TAX AND MOTOR-VEHICLE REVENUES TO THE GENERAL FUND AND PRORATED IN PROPORTION TO NET RECEIPTS FROM THESE IMPOSTS NOT OTHERWISE DEDICATED.

16/ EXPENDITURES FOR HIGHWAY PURPOSES HAVE BEEN CREDITED AGAINST PAYMENTS OF MOTOR-FUEL TAX AND MOTOR-VEHICLE REVENUES TO THE STATE GENERAL FUND AND PRORATED IN PROPORTION TO NET RECEIPTS FROM THESE IMPOSTS NOT OTHERWISE DEDICATED.

17/ MOTOR-FUEL TAX AND MOTOR-VEHICLE REVENUES ARE PAID INTO HIGHWAY ACCOUNT OF STATE GENERAL FUND, AND ARE USED FOR ANY PURPOSE, INCLUDING HIGHWAYS. EXPENDITURES FOR HIGHWAY PURPOSES HAVE BEEN CREDITED AGAINST RECEIPTS FROM MOTOR-FUEL TAX AND MOTOR-VEHICLE REVENUES, PRORATED IN PROPORTION TO NET RECEIPTS FROM THESE IMPOSTS.

U. S. GOVERNMENT PRINTING OFFICE: 1942

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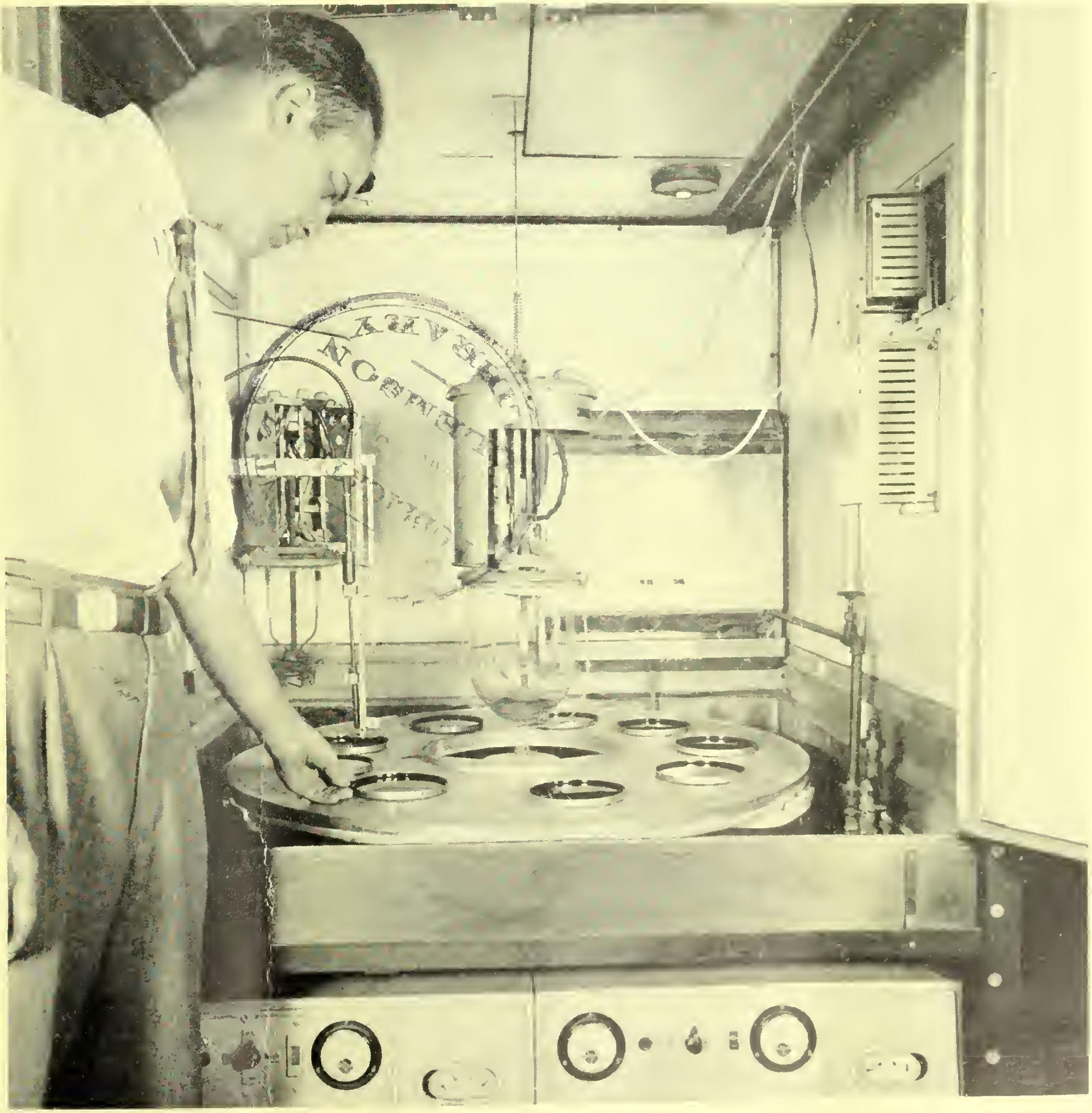

SUBJECTING TAR SAMPLES TO ACCELERATED WEATHERING TEST

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PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

Issued by the

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D. M. BEACH, *Editor*

Volume 23, No. 7 January–February–March 1943

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Commissioner of Public Roads, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

THE SULFONATION INDEX TEST FOR ROAD TARS[1]

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by R. H. LEWIS, Senior Chemist, and W. J. HALSTEAD, Associate Chemist

The erratic behavior of some tar products in localities where tars have given excellent service for many years has led to specifications limiting the amount of paraffinic and naphthenic hydrocarbons that they may contain. The amount of these materials is controlled by maximum requirements for the sulfonation indexes of two fractions of the tar distillate. The sulfonation indexes are determined by treating portions of the 0–300° C. fraction and the 300°–355° C. fraction of the distillate with 37 normal sulfuric acid under specified test conditions. The volume of the unsulfonated residue expressed in millimeters in each fraction per 100 grams of tar is reported as the sulfonation index for that fraction.

This report briefly discusses the types of hydrocarbons present in asphaltic and tar products. It suggests modification in the test method for determining the sulfonation index that will insure more reproducible and more accurate results. It demonstrates that the solubility of tar distillates in dimethyl sulfate and diethyl sulfate can not be used as a quantitative test for the paraffinic and naphthenic hydrocarbons.

A study of the relationship between the sulfonation indexes and the weathering properties of tars was made by exposing 1/8-inch films of the tar and compressed sand-tar mixtures to the accelerated weathering conditions of the weatherometer. These studies indicate that tars with the higher sulfonation indexes show the greater hardening which is caused by factors other than loss of volatile material, and also that those materials with a high sulfonation index on the 300°–355° C. fraction develop a lower bonding strength than those which have low sulfonation indexes for this fraction. However, the abnormal behavior of some of the tars during the laboratory tests and the wide differences in the characteristics of tars meeting the same specification indicate that a definite correlation of the sulfonation index with actual road behavior will be difficult to obtain.

IN RECENT YEARS several States have reported unsatisfactory service behavior with certain road tars which in all respects met the requirements of the controlling specifications. Priming material did not set up in normal curing time, delaying construction of the finished surface. The residual tars became greasy, showed evidence of separation or nonhomogeneity, and did not adequately hold the cover stone, necessitating early replacement of the road surface.

A study of these unusual conditions in areas where tar products had given excellent service for many years indicated that high percentages of paraffinic hydrocarbons were present in some of the tars that had not proved satisfactory. It was believed that in most cases this condition resulted from the use of petroleum products as a flux in the manufacturing process. However, it has been indicated also that some road tars derived from water-gas tar contain sufficiently high amounts of paraffinic material to affect adversely the quality, binding characteristics, and weathering properties of the tars.

In order to improve the performance of these road tars, it was recommended that the percentage of distillate at the various cutting temperatures used in the distillation test be more closely specified and that a maximum requirement for a sulfonation index of the distillation fractions, 0° to 300° C. and 300° to 355° C., be included in the specifications for road tars of the cold application type. The requirements for grades RT–1 to RT–6, inclusive, as proposed by some of the tar producers to meet this situation, and the standard specifications of the American Association of State Highway Officials, are given in table 1.

The specifications suggested by the producers were in use in some localities during 1940. Some users added the proposed limits for sulfonation index to the existing A. A. S. H. O. specifications; others changed the requirements for the sulfonation index on the 300° to 355° C. fraction from not more than 1.0 to not more than 1.5. As is noted in table 1, the A. A. S. H. O. now includes an optional requirement for the sulfonation index in its Specification M 52–42. This requirement applies to the grades RT–1 through RT–6 and the limits for the 0° to 300° C. fraction are the same as those proposed by some producers (Specification A, table 1) while the limit for the 300° to 355° C. fraction has been increased to not more than 1.5.

Although the sulfonation test has been used in the analysis of tar distillates for many years, it has not been employed extensively in the routine examination of road tars. This study of the sulfonation index test and the effect of the unsulfonated portion of the tar on its behavior under laboratory accelerated weathering conditions may, therefore, prove of interest to the users of road tars.

FOUR CLASSES OF HYDROCARBONS PRESENT IN BITUMINOUS MATERIALS

The significance and limitations of the sulfonation index test cannot be readily understood without some knowledge of the types of hydrocarbons that are constituents of both petroleum and tar products and which vary in quantity, depending on the source of the base material and the process used in refining. There are generally four large classes or divisions of hydrocarbons commonly present in petroleum and tars of all types. They are: (1) The paraffinic, which are the saturated, straight chain group; (2) the naphthenic, which are cyclic saturated compounds; (3) the olefinic, which are the unsaturated, straight-chain group; and (4) the aromatics, which are the derivatives and homologues of benzene.

Crude petroleum contains all of these groups in various proportions but generally the greatest percentage of hydrocarbons present is paraffinic or naphthenic. Products from the cracking of petroleum or petroleum distillates contain high percentages of the unsaturated or olefinic group and the aromatics.

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[1] Also known as the sulfonation factor.

caused by foaming during the shaking. It also was necessary to use less than 10 grams of distillate in some cases where the volume of unsulfonated residue was large and exceeded 2 milliliters for the standard weight of sample. The use of a smaller sample for the 300° to 355° C. distillate was also found to be most convenient for certain materials as it eliminated the necessity of making two distillations to this higher temperature in order to obtain 10 grams of distillate.

Subsequent to the completion of tests to be reported later (table 6), it was found that the producers' method failed to give reproducible results in some cases, especially when the amount of unsulfonated residue was large. Accordingly, a brief study was made of the effect of variations in the test procedure on the value of the sulfonation index.

THREE TARS HAVING HIGH, MEDIUM, AND LOW SULFONATION INDEXES STUDIED

Three tars were used for this study: A, having high sulfonation indexes; B, having medium indexes; and C, having very little unsulfonated residue. Sufficient distillate for each tar was collected and combined so that all the tests for each distillate were made on portions of the same material. The properties of the distillates are given in table 2. The method as shown above was used with variations in the procedure as listed in table 3.

Table 2.—*Properties of distillates used for studying the effect of various procedures on the results of sulfonation index test*

Sample identification	Distillate up to 300° C.			Distillate between 300° and 355° C.		
	Percentage of sample	Specific gravity at 25° C.	Index of refraction at 38° C.	Percentage of sample	Specific gravity at 25° C.	Index of refraction at 38° C.
A	33.3	0.949	1.543	15.2	1.024	1.597
B	31.5	.973	1.562	13.9	1.055	1.619
C	34.6	1.023	1.589	11.3	1.113	(1)

[1] Test not made; sample solid at 38° C.

Table 3.—*Variations in test procedure for determining sulfonation index*

	Procedure		
	No. 1	No. 2	No. 3
Amount of 37N acid in each portion ..... milliliters..	10	10	10
Number of portions added immediately	3	2	2
Total number of portions added	3	3	2
Time of heating before final addition of acid ...minutes..	0	30	0
Total time of heating ..... do....	60	60	60
Time of shaking after initial portion of acid .....do...	2	2	2
Time of shaking after each other portion of acid..seconds..	30	30	30
Time of shaking after each 10 minutes heating .....do....	30	30	30

Procedure No. 1, with only a slight change in the amount of shaking, is the same as in the producers' method; procedure No. 2 is that used for the tests reported later (table 6); and procedure No. 3 is a further modification that reduces the danger of the sample foaming over. In addition to these modifications in technique, the weights of samples were also varied. The results of these tests are shown in tables 4 and 5. Table 4 shows the results for the distillates up to 300° C. and table 5 shows the results for the 300° to 355° C. fraction.

For sample A, the results for the distillate up to 300° C. show wide variations for the different procedures, values for the sulfonation index ranging from a minimum of 6.3 to a maximum of 8.5. The values of the sulfonation index of this sample show that, in general, procedure No. 1 gives slightly higher values than does procedure No. 2 for the same weight of sample.

Table 4.—*Effect of various procedures on the results of the sulfonation index test for tar distillation up to 300° C.*

Test No.	Approximate weight of sample	Procedure No.	Volume unsulfonated	Sulfonation index
DISTILLATE FROM SAMPLE A				
	Grams		*Percent*	
1	10.0	1	23.1	8.1
2	7.5	1	22.8	8.0
3	5.0	1	20.7	7.3
4	10.0	2	22.5	7.9
5	7.5	2	21.3	7.5
6	5.0	2	19.5	6.8
7	10.0	3	21.7	7.6
8	5.0	3	18.8	6.6
9	5.0	3	19.6	6.9
10	5.0	3	19.0	6.6
11	5.0	3	18.4	6.5
12	5.0	(1)	18.0	6.3
13	5.0	(2)	20.5	7.2
14	5.0	(3)	24.1	8.5
DISTILLATE FROM SAMPLE B				
15	10.0	1	10.4	3.4
16	10.0	2	9.9	3.2
17	7.5	2	8.4	2.7
18	5.0	2	7.0	2.3
19	5.0	3	6.7	2.2
20	5.0	3	7.2	2.3
21	5.0	3	6.8	2.2
22	5.0	(1)	6.8	2.2
23	5.0	(4)	10.4	3.4
DISTILLATE FROM SAMPLE C				
24	10.0	2	1.2	0.4
25	7.5	2	1.0	.3
26	5.0	2	.8	.3
27	5.0	3	.8	.3

[1] Procedure 3 but with shaking at 5-minute intervals.
[2] Procedure 3 but with shaking at 15-minute intervals.
[3] Procedure 3 but with 10 seconds shaking at 10-minute intervals.
[4] Only 10 milliliters of 37N H_2SO_4 used.

Table 5.—*Effect of various procedures on the results of the sulfonation index test for the distillates between 300 and 355° C.*

Test No.	Approximate weight of sample	Procedure No.	Volume unsulfonated	Sulfonation index
DISTILLATE FROM SAMPLE A				
	Grams		*Percent*	
1	10.0	1	15.4	2.3
2	7.5	1	13.8	2.1
3	5.0	1	13.0	1.9
4	10.0	2	16.4	2.4
5	7.5	2	15.4	2.3
6	5.0	2	13.9	2.1
DISTILLATE FROM SAMPLE B				
7	7.5	2	7.7	1.0
8	5.0	2	7.4	1.0
9	5.0	3	6.5	.9
10	5.0	3	6.9	.9
11	5.0	3	7.0	.9
DISTILLATE FROM SAMPLE C				
12	10.0	2	(1)	(1)
13	7.5	2	(1)	(1)
14	5.0	2	(1)	(1)
15	5.0	3	(1)	(1)

[1] Trace.

The results also vary in order of the weight of sample of distillate, the 10-gram sample giving the highest result and the 5-gram sample giving the lowest.

For tests made by procedure No. 3, test No. 7 using 10 grams of distillate gives a considerably higher result than do any of the tests with this procedure in which 5 grams of distillate were used (tests Nos. 8–11). This indicates that 20 milliliters of 37N acid are insufficient for complete reaction of the 10-gram sample. Consequently, procedure No. 3 was used only with a 5-gram sample in all other determinations.

Procedure No. 3 with a 5-gram sample generally gives lower results than either of the other methods and the results check well. Tests Nos. 12, 13, and 14 illustrate the importance of shaking. In test No. 12 the sample was shaken at 5-minute intervals and gave a value of 6.3 for the sulfonation index. In test No. 13 the sample was shaken at 15-minute intervals and gave a value of 7.2. In test No. 14 the sample was shaken 10 seconds after each addition of the acid instead of 2 minutes after the first addition and 30 seconds after the second, and during the heating period it was shaken at 10-minute intervals for 10 seconds. The value obtained for the sulfonation index was 8.5. These values indicate that thorough shaking is necessary for complete sulfonation.

The results on the distillate up to 300° C. for sample B also show the same general variations as those for sample A. Procedure No. 1 and procedure No. 2, using larger samples, give higher results than those for the 5-gram sample for procedure No. 2 and procedure No. 3. Test No. 23 was made using only 10 milliliters of 37N H_2SO_4 on a 5-gram sample and the result was high.

The tests on distillate up to 300° C. for sample C show the same order of results, but the amount of unsulfonated residue is so small that the experimental error in reading the volume is practically equal to the differences obtained.

The results on the distillates between 300° and 355° C. show less variation for the different methods than do those for the distillates up to 300° C., indicating that the reactions take place more easily for this fracton and that the variations in test procedure are of less importance.

Since it is known that some of the aromatic compounds (benzene and its homologues) are difficult to sulfonate and since the strength of the acid used in the test is insufficient to produce an appreciable reaction with the paraffinic and naphthenic hydrocarbons, it is most likely that the test procedure which gives the smallest amount of unsulfonated residue is the most accurate. Therefore, it is indicated from these tests that procedure No. 3 will give more accurate results than either of the other two methods.

MODIFIED METHOD OF TEST PROPOSED

In view of these results, it is recommended that the method of test be modified to provide for a 5-gram sample of distillate and for manipulation as outlined in procedure No. 3. It is believed that these modifications will insure more complete sulfonation of the aromatic hydrocarbons and give better reproducibility than do the former practices. The proposed method is as follows:

PROPOSED METHOD OF TEST FOR SULFONATION INDEX OF ROAD TAR

A. SCOPE

1. This method of test is intended for use in determining the sulfonation index of the total distillate to 300° C. or of the fraction of the distillate from 300° C. to 355° C., obtained by means of the A. A. S. H. O. Standard Method of Test for Distillation of Tars and Tar Products, T 52–35.

B. SOLUTIONS REQUIRED

2. (*a*) *Sulfuric acid (37N).*—Prepare 37N H_2SO_4 by blending reagent-grade fuming and concentrated sulfuric acid to 98.61 ± 0.2 percent H_2SO_4, as determined by titration.

(*b*) *Sulfuric acid (specific gravity 1.84).*

C. APPARATUS

3. The following apparatus is required:

(*a*) *Test bottles.*—The test bottles shall be made of good quality glass and shall be 6-inch, 18 grams, either 8 percent or 10 percent Babcock milk test bottles. The capacity to the base of the neck shall be 45 to 50 milliliters. The graduated portion of the bottle shall contain 1.60 ± 0.025 milliliters for the 8 percent test bottle and 2.00 ± 0.025 milliliters for the 10 percent test bottle (at a temperature of 77° F. (25° C.)). The 8 percent test bottle shall be graduated in 8 major divisions, each major division being further divided in 10 subdivisions, and the 10 percent test bottle shall be graduated in 10 major divisions, each major division being further divided into 5 or 10 subdivisions. Each line for the major divisions shall extend at least three-fourths of the distance around the neck and be numbered from the bottom 1, 2, 3, etc. Within the range from 0–8 for the 8 percent test bottle and 0–10 for the 10 percent test bottle, the maximum error in volume shall not be greater than 0.025 milliliter. The graduation marks shall be clear and fine, not more than 0.3 millimeter in width. The bottom of the bottles shall have a ground area of at least 2 square centimeters for numbering.

(*b*) *Water baths.*—Two water baths, as follows: A water bath maintained at 77 ± 0.5° F. (25 ± 0.3° C.) and of such depth that when a test bottle is immersed, the upper level of its contents is below the surface of the water.

A water bath maintained at 208° to 212° F. (98° to 100° C.) and of sufficient depth to permit complete immersion of the body of the test bottle.

(*c*) *Centrifuge.*—A centrifuge capable of whirling two or more test bottles filled with acid at a speed of 1,000 revolutions per minute. The centrifuge shall be of good design and rugged construction so that it may be operated without danger.

(*d*) *Burette.*—One 50-milliliter burette graduated in 0.1-milliliter divisions.

D. PROCEDURE

4. (*a*) 5 ± 0.1 grams of the distillate, or fraction of distillate, shall be weighed into the test bottle. If the distillate contains solid matter, the distillate shall be warmed in a hot water bath, with stirring, until the solid matter has melted before the sample for testing is taken.

(*b*) Then 10 milliliters of 37N H_2SO_4 shall be slowly added to the test bottle from the burette in such a way as to wash down any oil remaining in the neck of the bottle. The test bottle shall then be shaken vigorously for 2 minutes. The temperature of this acid-distillate mixture shall not be allowed to approach 212° F. (100° C.) as indicated by the bottle becoming too warm to touch, or by the contents foaming excessively. The test bottle may be cooled in ice water, if necessary. However, if the distillate contains solid matter that does not readily disperse in the acid, it may be necessary to warm the distillate-acid mixture in the hot water bath to liquefy the solid matter.

(*c*) An additional 10 milliliters of 37N H_2SO_4 shall be added as outlined in paragraph (b) and the bottle shaken vigorously for 30 seconds. The test bottle shall then be placed in the water bath at 208° to 212° F. (98° to 100° C.).

(*d*) After the test bottle has been in the bath for 10 minutes, it shall be removed, shaken vigorously for 30 seconds, and replaced immediately in the water bath at 208° to 212° F. (98° to 100° C.). If the sample boils or foams over at this stage, it shall be discarded and the test repeated.

(*e*) The procedure outlined in paragraph (*d*) shall be repeated for a total of six 10-minute immersions and shakings. After the last shaking the test bottle shall be allowed to cool.

(*f*) After cooling the test bottle to approximately room temperature, sufficient H_2SO_4 (specific gravity 1.84) shall be added to the contents of the test bottle to raise the liquid level in the neck to near the top of the graduation. The test bottle and its contents shall then be placed in the centrifuge and whirled at a speed of approximately 1,000 revolutions per minute for 5 minutes. The bottle shall then be removed from the centrifuge and placed in the water bath at 77° F. (25° C.) so that the upper level of its contents is below the surface of the water. After 10 minutes the test bottle shall be removed from the bath and the volume of oil read to within one-tenth of a major division. The centrifuging shall be repeated until the volume of oil is constant.

(*g*) Generally the unsulfonated residue is a clear transparent oil. Occasionally some white or even dark-colored solid may be present. In case solid material appears, which cannot be melted in the water bath without causing overflow in the bottle, a new test should be made. This is to be done in the same manner as previously described except that when the bottle is removed from the hot bath it should be filled immediately with H_2SO_4 (specific gravity 1.84) to a point below the top of the graduations to allow room for expansion. The bottle should then be whirled in the centrifuge for 3 minutes and again placed in the boiling water bath for 5 minutes. This cycle shall be repeated until the volume of the oil is constant. The final reading of the volume shall be made while the test bottle is immersed in the hot water bath (98° to 100° C.) and any material which is solid at this temperature (resins) shall not be included as unsulfonated residue.

(*h*) After a constant oil volume has been obtained by centrifuging, the reading of the bottom of the oil level shall be subtracted from the reading of the top and the difference, in terms of the major divisions, multiplied by 0.2. The value so obtained is the milliliters of unsulfonated residue in the sample. The sulfonation index is obtained by dividing the milliliters of unsulfonated residue by the weight of the sample in grams and multiplying this quotient by the percentage of distillate by weight in the tar. Thus, the sulfonation index is the milliliters of unsulfonated residue per 100 grams of tar. It shall be reported to the nearest 0.1.

E. PRECAUTIONS

5. (*a*) It is extremely important that all glassware used in this test shall have been thoroughly cleaned and dried before use.

(*b*) The required amount of vigorous shaking is very important and in no case should the designated number of shakings or the time of shaking be decreased below the limits set. It is also important that the shaking shall be such that the distillate will be completely dispersed through the acid at the conclusion of each shaking period. If excess foaming results from the shaking after any heating interval, the bottle should be cooled for 15 or 20 seconds in a cold water bath, after which the shaking can usually be continued without the contents foaming.

(*c*) The rate of whirling in the centrifuge may be decreased to avoid breakage of the test bottles. In all cases, however, the centrifuging shall be continued until a constant reading of the volume of unsulfonated residue is obtained.

(*d*) If a separate layer of sulfonation residue cannot readily be observed a more definite line of demarkation may be established by carefully adding one or two drops of water to the bottle. This sometimes calls attention to solids which, because of their color, might escape notice.

The strength of the 37N acid used in this investigation of the sulfonation index test was determined volumetrically as follows:

Approxiately 2 grams of the acid were added from the burette to a weighing bottle. This was done as quickly as possible and the bottle stoppered immediately to prevent the absorption of water. The sample was weighed to the nearest 0.1 milligram, and it was then diluted with distilled water, the volume being made up to 500 milliliters. Twenty-milliliter portions of this solution were then titrated against approximately 0.1N NaOH which had been standardized against 0.1N H_2SO_4. The strength of the dilute acid had been determined gravimetrically by precipitation with barium chloride and weighing as barium sulfate.

For use in the producers' method, acid which contains 80.07 percent SO_3 is specified and no tolerances were given. However, it is considered that a reproducibility of results better than ±2 parts per thousand with this method cannot be obtained since this represents a burette reading of approximately 0.05 milliliter for the weights and strengths of solution mentioned. In view of this fact no effort was made to control the strength of the acid any closer than 80.1 ± 0.2 percent SO_3 (98.6 ± 0.2 percent H_2SO_4). A refilling burette was attached to the acid bottle so that it was unnecessary to open it to the atmosphere at any time after the initial standardization. The air entering the bottle was passed through a drying tube of $CaCl_2$ to remove any moisture, and these precautions insured a satisfactory degree of constancy in the strength of the acid.

MAJORITY OF TARS TESTED OF COLD-APPLICATION TYPE

Table 6 shows the test characteristics of the tars studied in this investigation. Samples 1 through 10 were submitted for routine tests prior to 1940 and were not required to meet specifications for the sulfonation index. Samples 11, 12, and 13 were submitted by one producer for a special study, sample 11 being a "bunker C" water-gas tar, sample 12 an unrefined vertical retort tar, and sample 13 a blend of these two which was refined and marketed as RT–4. Samples 14 and 15 were submitted to meet specifications requiring a sulfonation index less than 7 on the distillate up to 300° C. and less than 1.5 on the distillate between 300° and 355° C. These two materials are from the same producer but differ in their physical properties. With the exception of sample 11, all these materials were of the cold-application grades, eight being RT–2, one RT–3, and five RT–4. Sample 11 had a consistency corresponding to RT–8.

In addition to these materials, which were studied more thoroughly, sulfonation tests were made on some samples received during the 1940–41 construction seasons. Samples 16, 18, 21, and 22 represent materials from producers or from local plants of the producers for which samples were not available in the original study.

Samples 23 through 30 represent materials supplied by one producer for one large project. Samples 23 and 24 were taken at the time of the initial delivery of the material, and samples 25 through 30 were taken later during the period of construction. The material was stored in a large tank and evidently was not agitated during the period of use, which covered several months.

In addition to the test results on the samples studied in this laboratory, data obtained by the Ohio Department of Highways in an investigation of the sulfonation index and presented to the Committee on Materials of the A.A.S.H.O. are included in this report.[7] The tars tested in Ohio were from a majority of the producers represented in the Public Roads Administration investigation, as well as other producers, and these supplementary data provide a more general picture of the tars produced in the several localities. These data also include interesting results on several types of asphaltic products.

The results of the tests made by the Ohio Department of Highways are shown in tables 7, 8, and 9. Table 7 shows a summary of the tests on a number of tars which were marketed in Ohio. Table 8 shows the sulfonation indexes for several types of slow-curing

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[7] Appendix VI to Minutes of Meeting of Committee on Materials held March 3–4, 1941.

Table 6.—*Physical and chemical properties of the tars examined*

Sample No.	Producer No.	Engler specific viscosity at 40° C.	Specific gravity at 25° C.	Bitumen	Distillate: To 170° C.	Distillate: 170° to 200° C.	Distillate: 200° to 235° C.	Distillate: 235° to 270° C.	Distillate: 270° to 300° C.	Distillate: 300° to 355° C.	Residue from distillation to 300° C.	Softening point of residue: Above 300° C.	Softening point of residue: Above 355° C.	Specific gravity of distillate at 38° C.: To 300° C.	Specific gravity of distillate at 38° C.: 300° to 355° C.	Index of refraction of distillate at 38° C.: To 300° C.	Index of refraction of distillate at 38° C.: 300° to 355° C.	Sulfonation index: To 300° C.	Sulfonation index: 300° to 355° C.	Volume un sulfonated residue in distillate: To 300° C.	Volume un sulfonated residue in distillate: 300° to 355° C.
				Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	° C.	° C.							*Percent*	*Percent*
1	1	9.7	1.103	91.89	1.0	[1] 8.8	-------	14.8	8.0	16.7	65.4	35.3	76.1	0.958	1.010	1.558	1.596	6.7	2.2	19.8	13.4
2	1	10.0	1.100	90.77	1.8	[1] 10.3	-------	13.2	7.3	15.2	66.7	34.6	73.7	.939	1.013	1.537	1.595	8.9	2.8	25.8	18.8
3	1	9.1	1.126	92.56	1.8	[1] 10.1	-------	11.0	7.7	13.9	68.5	40.3	73.5	.963	1.046	1.561	1.619	3.8	1.4	12.1	10.6
4	2A	11.7	1.148	93.89	.7	[1] 12.5	-------	13.0	6.3	11.3	65.4	45.6	69.5	1.012	1.092	1.586	[2] 1.644	.7	(3)	2.2	(3)
5	2B	17.7	1.125	96.48	.6	[1] 9.0	-------	11.1	6.8	13.9	71.8	37.2	60.8	.960	1.074	1.548	[2] 1.628	4.4	.3	15.5	2.3
6	2B	23.2	1.135	96.52	.7	[1] 7.5	-------	11.9	6.2	14.5	73.5	37.6	64.4	.969	1.074	1.551	1.630	4.4	.3	16.3	2.2
7	1	12.1	1.118	95.32	1.4	[1] 8.2	-------	9.7	7.0	17.9	69.8	49.2	83.0	.963	1.040	1.564	1.615	1.6	.7	5.9	4.1
8	1	21.6	1.116	90.15	.7	.7	4.3	14.5	13.2	14.7	70.6	32.4		.949	1.019	1.548	1.598	10.4	2.5	29.8	17.5
9	3A	24.7	1.142	93.09	.3	.7	10.2	11.4	15.2	13.6	72.1	10.4		.950	1.090	1.554	[2] 1.640	9.8	.3	24.8	2.4
10	2C	29.9	1.174	93.07	.6	1.9	8.0	10.6	6.4	13.9	72.1	45.1		1.027	1.102	--	--	(3)		(3)	
11	4	(4)	1.177	92.55	.1	.8	3.1	6.6	6.2	15.1	82.5	51.8	83.5	.995	1.057	1.589	1.634	.4	.06	2.4	.4
12	4	8.5	1.091	95.01	.8	1.4	6.8	11.1	8.9	18.4	69.0	31.5	61.8	.960	1.035	1.542	1.598	4.9	2.0	16.4	11.3
13	4	35.1	1.112	95.21	.6	1.0	3.8	10.8	9.2	17.4	74.7	33.4	64.0	966	1.036	1.551	1.603	3.9	1.6	15.0	9.6
14	1	10.4	1.126	92.02	1.1	1.3	7.6	13.9	8.1	14.2	66.2	44.1	79.8	.975	1.046	1.570	1.621	3.6	.9	11.1	6.7
15	1	10.8	1.118	91.70	3.2	4.6	7.2	8.2	6.2	15.9	67.4	51.0	91.0	.947	1.037	1.551	1.615	3.3	1.4	10.7	9.2
16	5A	11.3	1.150	95.27	3.4	[1] 5.6	-------	9.7	8.1	18.2	71.8	37.7				--	--	.6	.03		
17	4	17.5	1.108	93.91	.2	1.2	5.5	11.3	8.4	15.1	70.1	38.1	--			--	--	3.8	1.6	--	
18	3B	18.2	1.143	95.23	.6	[1] 5.6	-----	12.4	10.6	-----	70.6	29.9	--			--	--	4.6	.5		
19	1	25.0	1.102	95.16	.2	.4	4.8	18.6	9.4	17.4	64.6	42.2	92.5	.939	1.029	1.537	1.613	10.5	1.3		
20	1	(5)	1.164	89.60	0	0	2.3	10.5	8.0	15.8	78.8	49.3	84.7	.961	1.034		--	4.2	1.4		
21	6	10.0	1.168	91.47	1.6	4.4	8.3	5.5	3.2	14.4	72.9	43.6	63.5	.980	(6)		--	.4	0		0
22	5B	12.1	1.120	95.78	1.5	1.2	6.3	10.9	8.0	15.3	71.1	43.8	80.0	.968	1.037	1.565	1.613	2.3	7	7.9	4.9
23	1	12.2	1.121	91.84	2.3	3.4	7.6	9.0	7.4		69.9	44.4		.948	--	--	----	3.2	1.4		
24	1	11.9	1.122	95.85	2.0	2.3	8.0	11.3	6.2	14.4	67.7	47.8	80.5	--	--	--	--	3.7	1.3		
25	1	10.5	1.122	92.28	1.0	.9	9.3	11.2	9.2	14.6	68.0	42.4						3.8	1.1		
26	1	[7] 5.0	1.086	91.13	3.2	4.7	10.1	14.2	7.4	9.3		44.6						5.3	.7		
27	1	11.6	1.116	91.76	4.1	4.4	8.2	7.7	6.9	11.1	68.0	48.2						4.7	1.3		
28	1	13.9	1.119	92.33	2.2	3.4	8.8	9.4	6.7	15.1	70.2	48.2						3.8	1.6		
29	1	9.6	1.110	91.45	2.2	3.7	10.5	10.2	5.8	15.1		57.8	--	--	--	----		5.9	2.1		
30	1	13.1	1.113	91.85	2.8	3.6	7.6	9.2	6.4	12.1	69.8	47.7	--	--	------		--	3.9	1.4		

[1] Percentage of distillate from 170° to 235° C.
[2] Liquid only.
[3] Trace.
[4] Float test at 32° C., 85 seconds.
[5] Float test at 32° C., 103 seconds.
[6] Fraction solid at 38° C.
[7] Sample contains 12.6 percent water.

Table 7.—*Test characteristics of road tars tested by Ohio Department of Highways*

Specification grade	Producer No.	Specific gravity at 25° C.	Total bitumen	Softening point	Total distillate to 300° C.: Percentage of sample	Total distillate to 300° C.: Sulfonation index	Distillate from 300° C. to 355° C.: Percentage of sample	Distillate from 300° C. to 355° C.: Sulfonation index
			Percent	° C.				
RT-2	5C	1.165	92.10	39.2	24.7	0.7	13.0	0
RT-2	5C	1.164	88.56	41.6	23.4	.8	13.2	0
RT-6	5C	1.174	95.69	37.6	21.5	.3	15.7	.1
RT-2	1	1.136	92.41	43.8	34.8	6.0	16.6	1.1
RT-2	1	1.123	92.23	47.0	34.8	6.3	15.7	1.4
RT-8	1	1.208	90.56	43.8	21.5	.7	28.3	.3
RT-6	7	1.168	91.37	53.5	21.7	.6	16.2	.1
RT-6	7	1.185	88.91	52.2	24.2	.5	13.7	0
RT-6	7	1.183	89.43	52.0	23.8	.5	18.3	0
RT-7	7	1.167	89.90	51.6	23.1	.8	15.8	.3
RT-6	7	1.184	89.45	47.2	24.0	.5	15.0	0
RT-6	7	1.167	91.19	48.5	26.1	.7	16.6	0
RT-6	7	1.166	91.37	45.0	25.3	.9	19.1	.4
RT-6	7	1.180	89.92	49.0	25.1	.6	------	---
RT-2	2D	1.135	96.36	42.8	29.5	.8	13.3	.1
RT-6	2D	1.189	94.78	53.6	22.0	.1	17.3	0
RT-5	2D	1.193	93.60	40.0	23.6	.2	17.2	.1
RT-6	2C	1.187	92.22	48.6	25.2	.9	--------	-----
RT-6	8	1.199	90.32	45.6	18.9	.2	16.4	0
RT-6	3C	1.217	89.15	39.2	18.7	.2	13.6	.1
RT-6	3C	1.201	92.75	39.8	19.9	1	14.4	.1
RT-2	3D	1.144	95.27	35.0	32.5	2.9	14.1	.3
RT-6	3E	1.207	88.84	48.4	19.4	0	10.9	0
RT-5	3C	1.201	93.83	41.4	19.4	0	13.0	0
RT-6	3D	1.200	90.49	39.0	20.6	.4	13.6	.1
RT-6	3E	1.205	91.29	41.0	19.3	.1	------	-
RT-4	9	1.163	96.84	33.2	27.0	.1	16.4	0
RT-4	9	1.164	95.06	35.0	28.9	.2	19.3	0
RT-2	10	1.161	95.63	32.0	25.8	.2	----	------

Table 8.—*Sulfonation indexes of various types of slow-curing liquid asphaltic road materials*

[Tests made by Ohio Department of Highways]

Sample identification	Specific gravity at 25° C.	Xylene equivalent	Total distillate to 300° C.: Percentage of sample	Total distillate to 300° C.: Sulfonation index	Distillate from 300° to 355° C.: Percentage of sample	Distillate from 300° to 355° C.: Sulfonation index
W	1.001	83-85	15.2	6.4	21.9	6.7
X	1.044	100+	20.8	3.9	23.2	4.4
Y	1.006	100+	17.8	7.8	19.1	6.1
Z	.917	0	12.7	11.4	36.1	27.7

Table 9.—*Effect of blending an asphaltic product with a tar on its test characteristics*

[Tests made by Ohio Department of Highways]

Test		RT-2	SC O	Blend [1]
Specific gravity at 25° C		1.165	1.052	1.118
Total bitumen soluble in CS_2	*percent*	92.10	99.68	97.02
Solubility in CCl_4	*percent*		98.68	
Amount of 100 penetration residue	*percent*		64.9	
Total distillate to 360° C.[2]	*percent*		31.0	
Float of residue at 50° C.	*seconds*		113	
Total distillate to 300° C.[3]	*percent*	24.7	20.8	26.
Softening point of residue	° C	39.2		35.0
Distillate from 300-355° C	*percent*	13.3	23.2	13.4
Sulfonation index, distillate to 300° C		.7	3.9	3.3
Sulfonation index, distillate 300-355° C		0	4.4	1.4

[1] Blend contains 65 percent of the RT-2 and 35 percent of the SC O
[2] Method A. S. T. M. D402-36.
[3] Method A. S. T. M. D20-30.

liquid asphaltic road materials and table 9 shows test results obtained with a tar of the RT–2 grade, one of the slow-curing asphaltic materials, and a blend of these two.

An examination of tables 6 and 7 will show that almost all of the tars having high sulfonation indexes were received from producer No. 1, while samples from plants A, B, and D of producer No. 3 had considerable unsulfonated residue in the portion of the distillate up to 300° C. but very little in the portion between 300° and 355° C. Some samples from producer No. 4, which contained vertical retort tar also had an appreciable amount of unsulfonated residue. The unsulfonated residues from the 300° to 355° C. fractions of these samples were peculiar in that they were solid at room temperature.

Table 8 serves to illustrate the amount of unsulfonated residue in various types of slow-curing liquid asphaltic road materials, and the effect of cracking on the sulfonation index. Those samples having high xylene equivalents have much lower indexes than do the materials which react negatively in the spot test. Table 9 is very interesting since it shows that tars having low sulfonation indexes can be blended with appreciable percentages of certain cracked petroleum products without exceeding the proposed limits for the sulfonation index.

SUBSTITUTE TESTS FOUND UNSATISFACTORY

Since the sulfonation index test requires the use of extensive laboratory equipment and is a somewhat complicated procedure requiring carefully controlled reagents, attention was given to the possibility of developing a substitute test that would give comparable results. In 1913, Reeve and Lewis made a study of the dimethyl sulfate test as a means for determining approximately the amounts of petroleum products in the tar-oil or tar-asphalt mixtures.[8] This test depends on the selective solubility of the various hydrocarbons in dimethyl sulfate in which the aromatic and unsaturated, or olefinic hydrocarbons, and their derivatives are considered to be completely soluble, and the paraffinic and naphthenic essentially insoluble. The test is made by shaking 6 milliliters of dimethyl sulfate with 4 milliliters of distillate in a 10-milliliter graduate and reading the volume of insoluble distillate which settles out. This amount is considered to be the paraffinic and naphthenic hydrocarbons. Since dimethyl sulfate is very poisonous some study has been made of the use of the less poisonous diethyl sulfate as a substitute.

J. N. Taylor [9] investigated the use of this material and developed a method whereby he used 5 milliliters of a petroleum oil (completely immiscible with diethyl sulfate), 5 milliliters of distillate, and 16 milliliters of diethyl sulfate. The petroleum oil was added to repress the solubility of the distillate in the diethyl sulfate, and by this method he found it possible to check closely with the amount of unsulfonated material in a synthetic mixture of petroleum oil and benzenes. He also found close agreement between this test and the sulfonation residue for the distillate obtained from "dips." In view of the success of these methods for the purposes of these authors, it was decided to investigate both these methods for use as a substitute for the sulfonation index test on tar distillates.

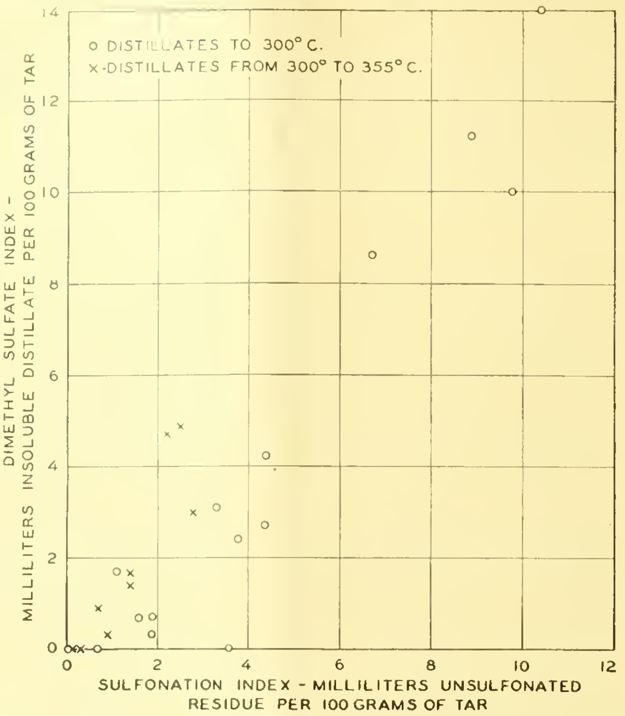

FIGURE 1. RELATION BETWEEN DIMETHYL SULFATE INSOLUBLE AND SULFONATION INDEX.

The dimethyl sulfate test was made on a number of the samples for which the sulfonation index was determined. The results of these tests are shown in table 10. These tests were made at room temperature as were those by Reeve and Lewis.[8] It is seen that there was not good agreement between the values obtained by the two methods, nor was there a definite relationship between the two values. This is illustrated in figure 1, in which the sulfonation index is plotted against the dimethyl sulfate index. An attempt was made to discover a relationship by varying the proportions of dimethyl sulfate to distillate and also by varying the temperature. These tests were made on the distillate from sample 2, and the results are shown in table 11.

TABLE 10.—*Comparison of distillate insoluble in dimethyl sulfate with amount of unsulfonated residue*

Sample No.	Distillate to 300° C.				Distillate from 300° to 355° C.			
	Sulfonation index	Dimethylsulfate index	Volume of unsulfonated residue	Volume insoluble in dimethyl sulfate	Sulfonation index	Dimethylsulfate index	Volume of unsulfonated residue	Volume insoluble in dimethyl sulfate
			Percent	*Percent*			*Percent*	*Percent*
1	6.7	8.6	19.8	25.0	2.2	4.7	13.4	32.0
2	8.9	11.2	25.8	32.0	2.8	3.0	18.8	28.0
3	3.8	2.4	12.1	7.0	1.4	1.4	10.6	10.0
4	.7	0	2.2	0	(1)	0	(1)	0
5	4.4	4.2	15.5	15.0	.3	0	2.3	0
6	4.4	2.7	16.3	10.0	.3	0	2.2	0
7	1.6	.7	5.9	3.0	.7	.9	4.1	5.0
8	10.4	14.0	29.8	40.0	2.5	4.9	17.5	34.0
9	9.8	10.0	24.8	25.0	.3	(1)	2.4	(1)
10	(1)	0	(1)	0	0	0	0	0
14	3.6	(1)	11.1	(1)	.9	.3	6.7	2.0
15	3.3	3.1	10.7	10.0	1.4	1.7	9.2	11.3

(1) Trace.

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[8] Application of the Dimethyl Sulfate Test for Determining Small Amounts of Petroleum or Asphalt Products in Tars. Journal Industrial Chemistry, vol. 5, No. 4, 1913.

[9] Diethyl Sulfate in the Examination of Hydro-Carbon Oils, by J. N. Taylor. Industrial and Engineering Chemistry, vol. 19, June 1927.

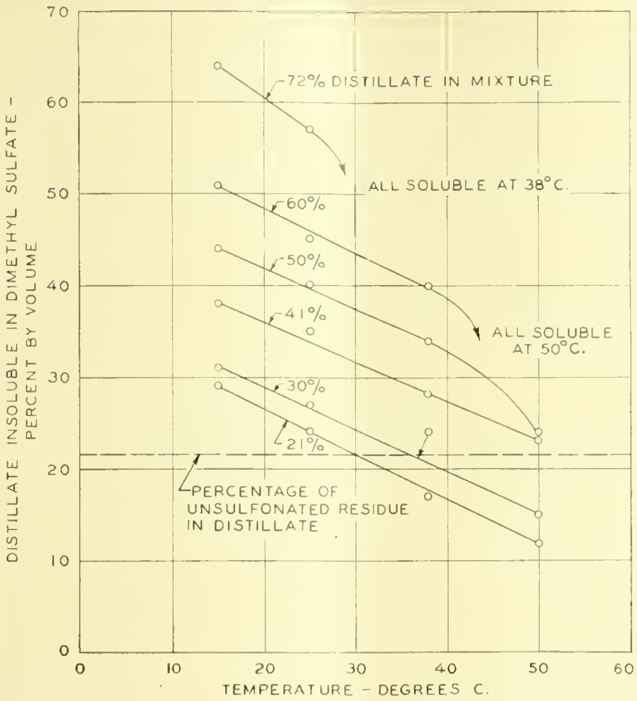

FIGURE 2.—VARIATION OF PERCENTAGE OF DISTILLATE INSOLUBLE IN DIMETHYL SULFATE WITH TEMPERATURE AND PROPORTIONS OF THE DISTILLATE IN THE TEST SOLUTION. DISTILLATE UP TO 300° C. FROM SAMPLE 2.

Figures 2 and 3 show the temperature plotted against the percent insoluble for each proportion. Figure 2 represents the data for the distillate up to 300° C. and figure 3 represents that for the distillate between 300° and 355° C. These results showed that variations in either the temperature or the proportions of dimethyl sulfate and distillate made large differences in the results. Tests were then made with distillate fractions from sample 2, as shown in table 12 and plotted in figure 4. These results again showed no general relationship and the percent insoluble which was equal to the amount of unsulfonated residue was obtained at widely different temperatures.

TABLE 11.—*The effect of temperature and proportion of solvent on the solubility of tar distillates in dimethyl sulfate*

DISTILLATE TO 300° C. (21.5 PERCENT BY VOLUME UNSULFONATED)

Volume of distillate in mixture	Volume of distillate insoluble in dimethyl sulfate at—				
	15° C.	25° C.	Room temperature [1]	38° C.	50° C.
Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
21	29	24	17	17	12
30	31	27	20	24	15
41	38	35	26	28	23
50	44	40	34	34	24
60	51	45	37	40	0
72	64	57	---	0	0

DISTILLATE FROM 300° TO 355° C. (15.2 PERCENT BY VOLUME UNSULFONATED)

20	25	20	15	17	17
29	31	26	24	22	17
40	35	30	30	28	23
51	41	39	36	33	26
60	57	48	43	39	31
70	60	54	45	44	0

[1] Room temperature approximately 32° C.

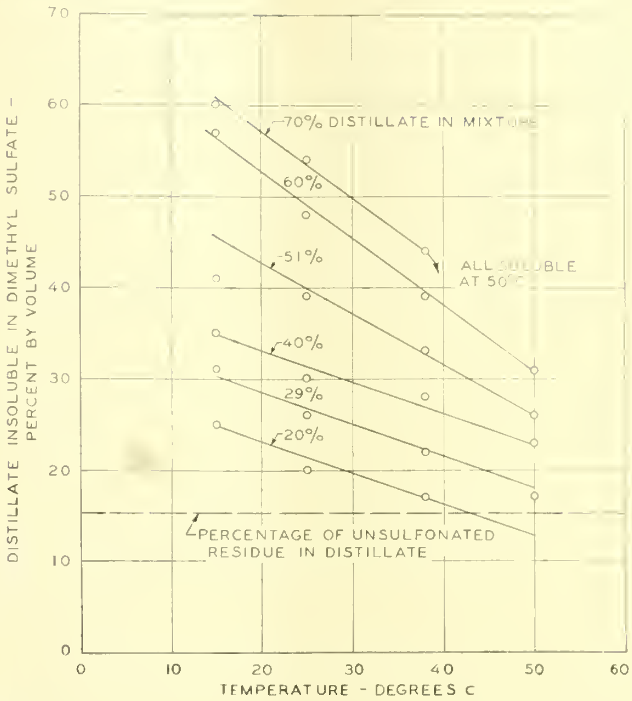

FIGURE 3.—VARIATION OF PERCENTAGE OF DISTILLATE INSOLUBLE IN DIMETHYL SULFATE WITH TEMPERATURE AND PROPORTIONS OF THE DISTILLATE IN THE TEST SOLUTION. DISTILLATE BETWEEN 300° AND 355° C. FROM SAMPLE 2.

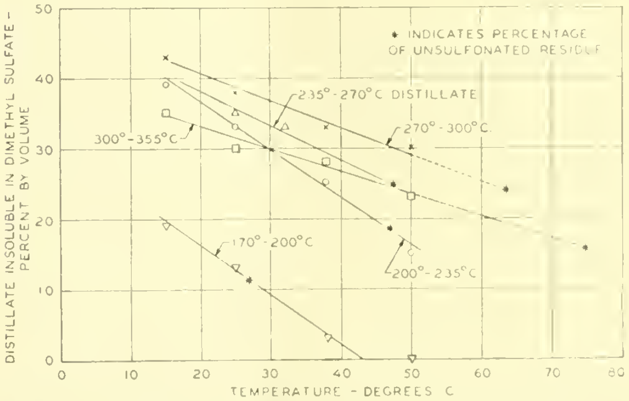

FIGURE 4.—VARIATION OF PERCENTAGE OF DISTILLATE INSOLUBLE IN DIMETHYL SULFATE WITH TEMPERATURE FOR VARIOUS FRACTIONS DISTILLED FROM SAMPLE 2.

TABLE 12.—*The effect of temperature on the solubility of the various distillation fractions of a tar in dimethyl sulfate*

Distilling temperature of fraction	Volume of unsulfonated residue	Volume of distillate insoluble in dimethyl sulfate at—				
		15° C	25° C.	Room temperature [1]	38° C.	50° C
° C.	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
170-200	11.4	19	13	8	3	0
200-235	18.5	39	33	25	25	17
235-270	24.9		35	33		
270-300	23.8	43	38	35	33	30
300-355	15.2	35	30	30	28	23

[1] Room temperature approximately 32° C

When diethyl sulfate was substituted for the dimet sulfate, using the same procedure as used for the methyl sulfate tests, no insoluble could be obtain even for distillates containing very high percentage of unsulfonated residue. Accordingly, Taylor's[9] method was tried. There was some difficulty in finding a petroleum oil that was completely immiscible with diethyl sulfate and at the same time had a low viscosity so that accurate volumetric measurements could be made. After experimentation, kerosene which had been sulfonated to remove all aromatic and unsaturated compounds was chosen as being the best material easily available. It was discovered, however, that there was a slight mutual solubility of the kerosene and diethyl sulfate. Thus, in order to obtain more accurate results, the kerosene was saturated with diethyl sulfate and the diethyl sulfate was saturated with kerosene. Using these precautions, tests similar to those with the dimethyl sulfate were made. The test temperature and proportions of solute and solvent were varied. The same general result was obtained and no relationship could be established.

These results show clearly that neither of these tests can be correlated with the sulfonation index test, nor can they be used as a test method for specification requirements. They are at best qualitative tests and can be used only as an indication of the presence of large amounts of paraffinic hydrocarbons.

RELATIONSHIP BETWEEN SPECIFIC GRAVITY OF DISTILLATE AND SULFONATION INDEX STUDIED

The relationship of the softening points of the distillation residues from both the distillation to 300° C. and the distillation to 355° C. to the amount of unsulfonated residue in the distillate was examined and no definite trend was found.

The specific gravity at 38° C. of the distillate was also determined and compared to the sulfonation index. No straight-line relation was found but there is a definite minimum value for the specific gravity above which the sulfonation index is always lower than the requirements in the proposed specifications. When the specific gravity is below this minimum, the sulfonation index is usually high. However, in some cases a low sulfonation index was found in conjunction with a low specific gravity.

These results are in accordance with the well-known ct that, for the same boiling point range, the specific avities of the aromatic hydrocarbons are higher than ose of the unsaturated compounds which, in turn, ve higher gravities than the paraffinic materials. he specific gravities of the naphthenic group, however, ll between those of the unsaturated and aromatic groups. Thus, the specific gravity of a mixture conaining the maximum specified amount of unsulfonated residue is greatest when this unsulfonated residue is naphthenic and the sulfonated portion is aromatic. This value of the specific gravity theoretically represents the minimum above which the sulfonation index is always less than the specified maximum. The converse is not true since the presence of the olefinic group which is readily sulfonated may lead to combinations of widely different percentages of unsulfonated hydrocarbons having the same specific gravity. Thus, if his "critical" specific gravity can be determined, a ause could be inserted in the specifications stating hat if the specific gravity of the distillate is greater han this value the material is acceptable. However, the specific gravity is less than this value, the sulfonaon index must be determined.

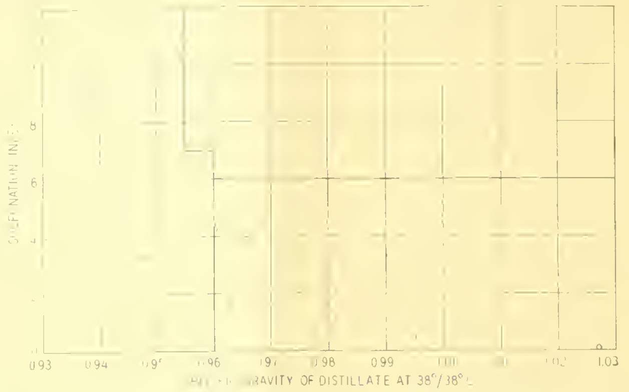

Figure 5.—Relation Between the Sulfonation Index and the Specific Gravity of the 0°–300° C. Fractions.

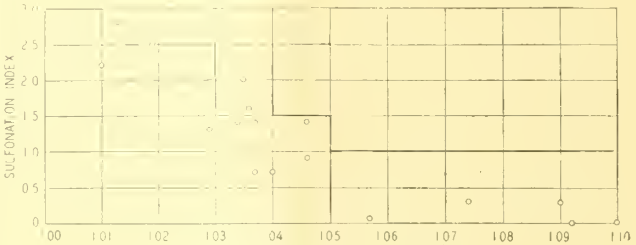

Figure 6. Relation Between the Sulfonation Index and the Specific Gravity of the 300°–355° C. Fractions.

Figures 5 and 6 show the sulfonation index plotted against the specific gravity of the distillate at 38°/38° C., figure 5 representing the data for the distillate up to 300° C. and figure 6 representing the data for the distillate between 300° and 355° C. These data were recorded in table 6. It is seen from the figures that for the materials tested, all samples of distillate to 300° C. which had a specific gravity at 38°/38° C. of 0.960 or more, had a sulfonation index less than 6. However, two of the distillates to 300° C. reported in table 6 had specific gravities lower than 0.960 and sulfonation indexes less than 6. Likewise, all samples having a specific gravity at 38°/38° C. of 0.955 or more, had sulfonation indexes less than 7. For the distillate from 300° C. to 355° C. this critical specific gravity is 1.05 at 38° C. for a sulfonation index of 1.0, and 1.04 at 38° C. for a sulfonation index of 1.5. However, 4 samples of the distillate between 300° C. and 355° C. which had specific gravities at 38° C. of less than 1.04 had sulfonation indexes less than 1.5, and 3 samples which had specific gravities at 38° C. of less than 1.05 had sulfonation indexes less than 1.0.

TARS AND SAND-TAR MIXES EXPOSED IN WEATHEROMETER

It should be pointed out that these figures represent the results for the samples tested by this laboratory and it is possible that tars from other sources, or those having unusually large amounts of distillate, might have slightly higher specific gravities than these values and still exceed the maximum values for the sulfonation index by small amounts. Nevertheless, it is apparent from the above discussion that under certain conditions it should be possible to eliminate the necessity of making a large number of sulfonation index determinations on road tars by establishing suitable "critical" values for the specific gravities of the distillates.

In order to correlate, if possible, the weathering properties of the tars with the amount of unsulfonated residue, two series of accelerated exposure tests were made in the weatherometer (see cover illustration) using thin films of the tars and molded sand mixes containing the tar.

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[9] Diethyl Sulfate in the Examination of Hydro-Carbon Oils, by J. N. Taylor, ndustrial and Engineering Chemistry, vol. 19, June 1927.

The tars were exposed in seamless flat tins 5½ inches in diameter and ⅝-inch deep which, when filled with 50 milliliters of tar, give the desired film thickness of ⅛ inch.

The mixes were made in the same proportions and with the same type of sand as in the work done by Lewis and Welborn in their study of road tars.[10] The mixes contained 16.6 percent by volume of tar, and the sand had a grading as shown in table 13.

Two sizes of cylinders were molded, one size 2 inches in diameter and 1 inch high for Hubbard-Field stability tests, and the other 1 inch in diameter and 1 inch high for toughness tests with the Page impact machine.

TABLE 13.—*Grading of sand in mixes on which accelerated exposure tests were made*

Passing sieve—	Retained on sieve—	Percent
No. 10	No. 20	3.7
No. 20	No. 30	10.3
No. 30	No. 40	18.1
No. 40	No. 50	21.3
No. 50	No. 80	36.6
No. 80	No. 100	6.1
No. 100	No. 200	3.2
No. 200		.7

The cylinders were molded at 3,000 pounds per square inch, the load being released immediately (single plunger method). The initial stability and impact were determined for all mixes immediately after molding.

The weatherometer was controlled at 60° C. and approximately 50 percent humidity, a dry cycle being used entirely. One group of samples, which consisted of one sample of tar and two of each size cylinder, was removed at the end of 60 hours exposure, the second after 200 hours, and the third after 400 hours. The percentage of loss, specific gravity at 25° C. and kinematic viscosity at several temperatures were determined for each of the residues from the tar samples, and the percentage of loss, stability at 25° C., and impact were determined for the sand-tar mixes.

In the first series, samples 1 through 10, with the exception of sample 5, were tested, and in the second series samples 11 through 15 were tested. It should be noted that for 20 hours between the 60- and 200-hour periods a blown fuse in the line operating the cooling fan caused the overheating of the samples in series 2 to approximately 95° C. For this reason these samples generally show a greater loss and more hardening than do those of the first series.

Table 14 shows the rate of loss of volatile matter for the tar samples in both series. These data are computed in three different ways for comparison. The loss of the sample, the loss of the distillate (based on the total distillate to 300° C.), and the proportion of the total loss are shown for each exposure period. Two results are given for each sample one, the value for the 400-hour sample at the end of each period, and the other the value for the sample removed from the weatherometer at the end of each intermediate time interval.

It is of interest to note two different types of behavior with these samples. Samples of one type formed a dull surface skin in a comparatively short time, and samples of the other type remained glossy until very near the end of the 400-hour test period. Samples 1, 2, 3, 7, 8, 14, and 15 exhibited the first type of behavior, and samples 4, 6, 9, and 10 exhibited the second type. Samples 11, 12, and 13 are of special interest since they are from known sources. Sample 11, the "bunker C" water-gas tar, had developed a dull and wrinkled skin by the end of 200 hours, while sample 12, the vertical retort tar, hardened greatly and developed a greasy black surface without any evidence of a surface skin. Sample 13, the blend of the two, exhibited both types of surface. It had a dull skin over most of the surface and wherever this skin had parted the exposed portion was very similar in appearance to the surface of sample 12.

TABLE 14.—*Rate of loss of volatile matter in thin-film exposures of tar samples in the weatherometer*

Sample No.[1]	Loss of sample after—			Loss of total distillate[2] after—			Proportion of total loss after—		
	60 hours	200 hours	400 hours	60 hours	200 hours	400 hours	60 hours	200 hours	400 hours
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1 a	28.3	28.7	29.0	82	83	84	98	99	100
b	28.4	29.0		82	84		98	100	
2 a	28.2	28.6	29.1	85	86	87	97	98	100
b	28.0	28.4		84	85		96	98	
3 a	25.0	25.2	25.6	79	80	81	98	98	100
b	25.0	25.4		79	81		98	99	
4 a	25.3	28.4	28.7	73	82	83	88	99	100
b	25.2	29.2		73	84		88	102	
6 a	22.4	23.7	24.1	85	89	91	93	98	100
b	22.5	24.2		85	91		93	100	
7 a	22.6	23.2	23.6	75	77	78	96	98	100
b	22.6	23.3		75	77		96	99	
8 a	25.5	25.9	26.2	87	88	89	97	99	100
b	25.5	25.9		87	88		97	99	
9 a	21.3	24.0	24.4	72	81	82	87	98	100
b	21.2	24.8		71	84		87	102	
10 a	20.1	22.5	22.7	72	81	81	89	99	100
b	20.4	20.0		73	72		90	88	
11 a	6.9	10.8	10.8	39	62	62	64	100	100
b	7.2	10.9		41	62		67	101	
12 a	34.8	40.8	43.3	112	132	140	80	94	100
b	28.9	34.9		93	113		67	81	
13 a	22.0	27.2	27.6	87	108	109	80	99	100
b	21.9	27.0		87	107		79	98	
14 a	27.1	31.2	31.2	80	92	92	87	100	100
b	26.7	30.7		79	91		86	98	
15 a	23.6	27.3	27.3	72	84	84	86	100	100
b	23.6	27.2		72	82		86	100	

[1] Figures in line a are for 400-hour residues at the designated time interval. Figures in line b are for the residues removed from the weatherometer at the designated time interval.

[2] As determined by A. S. T. M. distillation test to 300° C.

The results of all the tests on the tar residues from the weatherometer are shown in table 15. The kinematic viscosity data were plotted on the A. S. T. M. viscosity-temperature chart (high range) in order to obtain the viscosity-temperature susceptibility (V. T. S.) coefficient, which is the tangent of the angle that the plotted line makes with the temperature axis. A difference of 1 degree in the angle makes a difference of approximately 0.07 in the V. T. S. coefficient. Thus it is seen from the values reported that generally all of the lines for each sample were parallel or very nearly so, and in all cases where there was a difference the V. T. S. coefficient was smaller for the longer exposure periods.

"HARDENING INDEX" OFFERS MEANS OF COMPARING DEGREE OF HARDENING

Since there is such a small difference in the susceptibility of the various residues, the viscosity-temperature lines may be considered parallel, which means that the difference between the double logarithms of the viscosities of two residues is a constant at different temperatures. The magnitude of this difference is also a measure of the amount of hardening in a sample. Thus, for comparing the viscosities of a group of samples, the double logarithm of the kinematic viscosity at the temperature desired, multiplied by 100 for convenience, may be considered a "viscosity coefficient" for that temperature, and the difference between the "viscosity coefficients" for any two residues from the same sample is a measure of the amount of hardening which has taken place. This viscosity coefficient at 50° C., and the difference between the coefficients for the 200- and 60-hour residues, and the difference between the 400- and 200-hour residues are also shown in table 15.

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[10] A Study of Road Tars, by R. H. Lewis and J. Y. Welborn. PUBLIC ROADS, vol. 17, No. 5, July 1936.

Table 15.—*Results of tests on the residues after exposure in the weatherometer*

Sample No.	Time exposed	Loss	Specific gravity at 25° C.	Kinematic viscosity at 35° C.	Kinematic viscosity at 50° C.	Kinematic viscosity at 65° C.	Kinematic viscosity at 82° C.	V. T. S. coefficient	Viscosity coefficient at 50° C.	Change in viscosity coefficient
	Hours	*Percent*		*Stokes*	*Stokes*	*Stokes*	*Stokes*			
1	60	28.4	1.169	6,070	265	38.2	....	0.93	64.6	----
	200	29.0	1.174		1,880	121	18.5	.93	72.2	7.6
	400	29.0	1.178		6,660	274	32.5	.93	76.5	4.3
2	60	28.0	1.179	9,580	415	51.2	......	.97	66.4	---
	200	28.4	1.183		2,680	152	22.4	.97	73.5	7.1
	400	29.1	1.186		10,100	491	43.6	.97	77.8	4.3
3	60	25.0	1.192	2,950	222	40.8	....	.90	63.8	----
	200	25.4	1.198	......	926	92.4	17.3	.87	69.6	5.8
	400	25.6	1.201		2,440	226	32.9	.87	73.1	3.5
4	60	25.2	1.220	2,870	175	23.1	...	1.03	62.8	-
	200	29.2	1.229		2,080	143	18.0	1.00	72.6	9.8
	400	28.7	1.233		3,750	253	30.0	.97	74.6	2.0
6	60	22.5	1.200	8,330	372	36.0	......	1.07	66.0	--
	200	24.2	1.207	......	1,760	114	13.3	1.03	72.0	6.0
	400	24.1	1.209	......	2,400	163	17.4	1.03	73.1	1.1
7	60	22.6	1.182	3,050	241	34.2	....	.93	64.2	----
	200	23.3	1.188		1,210	121	20.9	.84	70.6	6.4
	400	23.6	1.191		2,650	245	37.7	.84	73.5	2.9
8	60	25.5	1.187	9,100	547	70.4		.93	67.6	
	200	25.9	1.189		2,390	195	29.9	.90	73.1	5.5
	400	26.2	1.189		5,810	430	50.0	.90	76.1	3.0
9	60	21.2	1.213	4,500	254	32.1	...	1.02	64.4	
	200	24.8	1.226	...	2,010	154	20.4	.95	72.5	8.1
	400	24.4	1.231		2,750	233	27.0	.95	73.6	1.1
10	60	20.4	1.224	5,620	279	31.0		1.07	64.8	--
	200	20.0	1.233		1,950	128	18.1	.95	72.3	7.5
	400	22.7	1.233		2,310	155	22.8	.93	72.9	.6
11	60	7.2	1.201	4,270	321	48.5	....	.92	65.4	---
	200	10.9	1.212		5,310	457	58.1	.88	75.8	10.4
	400	10.8	1.214		8,790	707	70.9	.88	77.4	1.6
12	60	28.9	1.153	1,550	105	13.6	......	1.09	60.4	--
	200	34.9	1.174		7,640	346	36.8	1.09	77.0	16.6
	400	43.3	1.186		175,000	4,880	258	1.04	86.0	9.0
13	60	21.9	1.162	6,010	352	39.1	.....	1.02	65.8	-
	200	27.0	1.179		7,890	456	46.5	1.98	77.1	11.3
	400	27.6	1.178		17,560	815	81.1	.98	79.6	2.5
14	60	26.7	1.188	2,170	181	29.9	.....	.90	62.9	----------
	200	30.7	1.196		2,960	271	36.5	.88	73.8	10.9
	400	31.2	1.200		6,420	536	65.3	.88	76.4	2.6
15	60	23.6	1.179	2,960	230	39.3	......	.92	64.0	----------
	200	27.2	1.190		4,330	362	50.2	.82	75.1	11.1
	400	27.3	1.193		6,730	560	72.6	.82	76.6	1.5

In some cases greater deviations from a straight line were obtained when the data were plotted on the A. S. T. M. viscosity-temperature chart than could be attributed to experimental error. This is believed to have been caused by a large amount of carbon particles in the residue which introduced plastic or nonuniform flow in the tube of the viscosimeter. The nonhomogeneity of these residues was also indicated by their dull or grainy appearance when stirred. In several instances, with the 60-hour residues from samples 2 and 7, there was a separation of a very small amount of clear oil on the top of the rising column of tar during the progress of the viscosity determination. This occurred during the first determination on the residue from sample 2 at 50° C. and at 65° C. For sample 7 it occurred during the first determination at 50° C. This separation always resulted in an abnormally low viscosity value which had to be discarded.

Generally these irregularities occurred with the residues from samples having high sulfonation indexes, while the residues from samples having low sulfonation indexes usually showed very little deviation from the straight line. However, sample 7, which had a low sulfonation index, exhibited these irregularities. In general appearance and test results this sample was very similar to those having high sulfonation indexes, while on the other hand sample 9, which had a very high sulfonation index on the distillate up to 300° C. but a low index on the distillate from 300° to 355° C., exhibited properties similar to the tars having low sulfonation indexes in both cases. These exceptions to the general behavior show how difficult it is to correlate the behavior of the road tar with its content of paraffinic hydrocarbons, or to set up a requirement for the sulfonation index that will definitely distinguish between tars that will be satisfactory in service and those that will be unsatisfactory.

Since tars lose their volatile constituents very rapidly in the weatherometer, the results of the tests shown in table 14 give no indication of the drying characteristics of the tars. In order to gain some knowledge of the changes in the physical properties of the tars during the drying period, a special study of samples 2, 3, and 4 was made. As is shown in table 6, these samples have almost the same viscosity, percentage of bitumen, and percentage of distillate to 300° C. The sulfonation indexes, however, differ greatly. Sample 2 has indexes of 8.9 for the distillate up to 300° C. and 2.8 for the distillate between 300° and 355° C., which exceed all the proposed specification limits. Sample 3 has indexes of 3.8 for the 0°–300° C. fraction and 1.4 for the 300° to 355° C. fraction, which are slightly lower than the maximum values proposed in some specifications. Sample 4 has very low sulfonation indexes—0.7 on the distillate up to 300° C. and a trace on the distillate from 300° to 355° C.

Three distillations were made for each of these tars, one up to 240° C., one to 270° C., and the third up to 300° C. In order further to compare the physical properties of these materials, the volume of the distillate and the liquid temperature were recorded for each 10° C. interval of vapor temperature. These distillations were made using the standard distillation apparatus. The specific gravity and the weight of the distillate from each distillation were determined and each residue was tested for kinematic viscosity at 50° C. and specific gravity at 25° C. The distillation data are plotted in figure 7 and it is seen that these samples have comparable volatility. The results of the tests on the distillates and the residues are shown in table 16. This table also includes a "hardening index" which is explained in the following discussion.

As stated previously, the viscosity coefficient may be represented by the double logarithm of the kinematic viscosity in centistokes multiplied by 100. This figure is an index of the consistency of the material at any

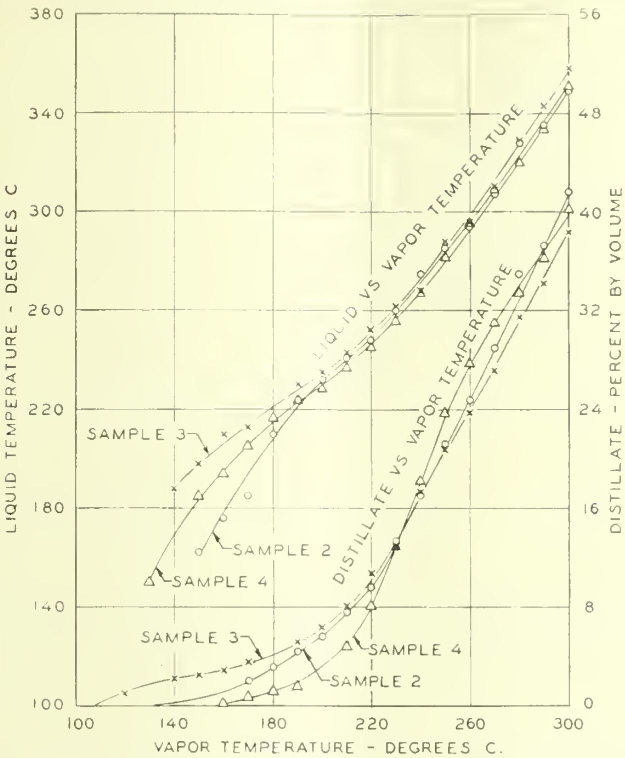

FIGURE 7.—RELATIONS BETWEEN THE LIQUID AND VAPOR TEMPERATURES, AND BETWEEN THE VOLUME OF DISTILLATE AND VAPOR TEMPERATURE.

particular time, but has no connection with the original consistency of the material or the percentage of loss. The hardening (increase in viscosity) of any material may be caused by evaporation losses or by chemical and physical changes within the tar itself, such as oxidation or polymerization, or by a combination of both factors. The first of these causes is a necessary property of liquid road materials and for rapid setting up should be as high as possible, while the hardening from changes within the structure of the tar itself should be as low as possible if the tar is to retain its life for a long period of time. Thus it is very desirable to establish the proportion of the hardening which should be attributed to evaporation losses and the proportion which should be attributed to other changes.

TABLE 16.—*Properties of the residues and distillates from selected tars distilled to various temperatures*

Sample No.	Highest distillation temperature	Specific gravity of residue at 25° C.	Specific gravity of distillate at 38° C.	Weight of distillate	Volume of distillate	Kinematic viscosity of residue at 50° C.	Viscosity coefficient of residue at 50° C.	Hardening index
	° C.			*Percent*	*Percent*	*Centistokes*		
2		1.100		0	0	53.8	21.1	
	240	1.137	0.912	14.3	16.7	564	41.0	1.32
	270	1.172	.930	26.5	31.0	6,320	58.0	1.28
	300	1.201	.940	31.9	40.7	105,000	70.1	1.32
3		1.126		0	0	41.5	21.7	
	240	1.168	.935	15.8	18.5	1,360	49.6	1.77
	270	1.193	.952	25.2	29.5	14,200	61.8	1.59
	300	1.217	.965	33.1	38.5	199,000	72.4	1.53
4		1.148		0	0	46.5	22.2	
	240	1.202	.992	19.5	22.5	1,770	51.2	1.49
	270	1.228	1.004	29.2	33.2	41,700	66.5	1.52
	300	1.245	1.013	35.6	40.6	831,000	77.2	1.55

TARS WITH HIGHEST SULFONATION INDEXES SHOW GREATEST "INTERNAL HARDENING"

The distillation test is conducted under conditions which should give the hardening caused by loss of volatile portions alone. It was found that when the viscosity coefficients at 50° C. were plotted as ordinates and the percentage of loss by weight in obtaining the distillation residues were plotted as abscissae, a close approximation to a straight line was obtained. The slope of this line represents the average amount of hardening for 1 percent of loss in the distillation test. It is a measure of hardening and is called the hardening index. However, in order to obtain a definite numerical value, the hardening indexes shown in tables 16 and 17 were calculated by the following equation:

$$HI=\frac{V_1-V_0}{W}$$

where

HI=hardening index,
V_1=viscosity coefficient at 50° C. of the residue,
V_0=viscosity coefficient at 50° C. of the sample, and
W=percentage of loss by weight in obtaining the residue.

It is noted that the values of the hardening index shown in table 16 are very nearly constant except for the distillation to 240° C. for sample 3, and this difference is probably due to experimental error.

The hardening characteristics of the three samples in the weatherometer were determined by exposing the tars in the usual ⅛-inch films for periods of 2, 4, 8, 16, 32, 64, and 128 hours. The residues from each of these periods were weighed to obtain the percentage of loss. The kinematic viscosity at 50° C. and the specific gravity at 25° C. were also determined. The hardening index for each residue was calculated. All these results are shown in table 17.

TABLE 17.—*Properties of the residues from selected tars exposed in the weatherometer*

Sample No.	Time exposed	Specific gravity of residue at 25° C.	Loss in weight	Kinematic viscosity at 50° C	Viscosity coefficient at 50° C.	Hardening index
	Hours		*Percent*	*Centistokes*		
2	0	1.100	0	53.8	24.1	
	2	1.128	11.9	285	39.0	1.25
	4	1.140	16.3	753	45.9	1.34
	7	1.151	20.4	1,930	51.7	1.35
	16.5	1.164	24.5	6,550	58.2	1.39
	32	1.172	27.0	22,500	63.9	1.47
	64	1.179	28.7	71,800	68.6	1.55
	128	1.181	28.6	220,000	72.8	1.70
	[1] 200	1.183	28.4	268,000	73.5	1.74
	[1] 400	1.186	29.1	1,010,000	77.9	1.85
3	0	1.126	0	44.5	21.7	
	2	1.152	11.5	264	38.4	1.45
	4	1.160	14.2	501	43.1	1.51
	7	1.169	18.2	1,280	49.3	1.51
	16.5	1.180	22.3	4,940	56.7	1.57
	32	1.187	24.4	12,800	61.3	1.62
	64	1.192	25.2	28,400	64.9	1.71
	128	1.195	25.2	53,200	67.5	1.81
	[1] 200	1.198	25.4	92,700	69.6	1.89
	[1] 400	1.201	25.6	241,000	73.1	2.01
4	0	1.148	0	46.5	22.2	
	2	1.180	10.8	265	38.4	1.50
	4	1.188	14.3	538	43.6	1.50
	7	1.197	17.6	1,260	49.1	1.53
	16.5	1.207	21.5	4,330	56.1	1.57
	32	1.211	24.2	13,700	61.7	1.63
	64	1.220	26.5	36,200	65.9	1.65
	128	1.224	28.4	133,000	71.0	1.72
	[1] 200	1.229	29.2	208,000	72.6	1.73
	[1] 400	1.233	28.7	375,000	74.6	1.83

[1] Results from tests made in series 1; also shown in table 15.

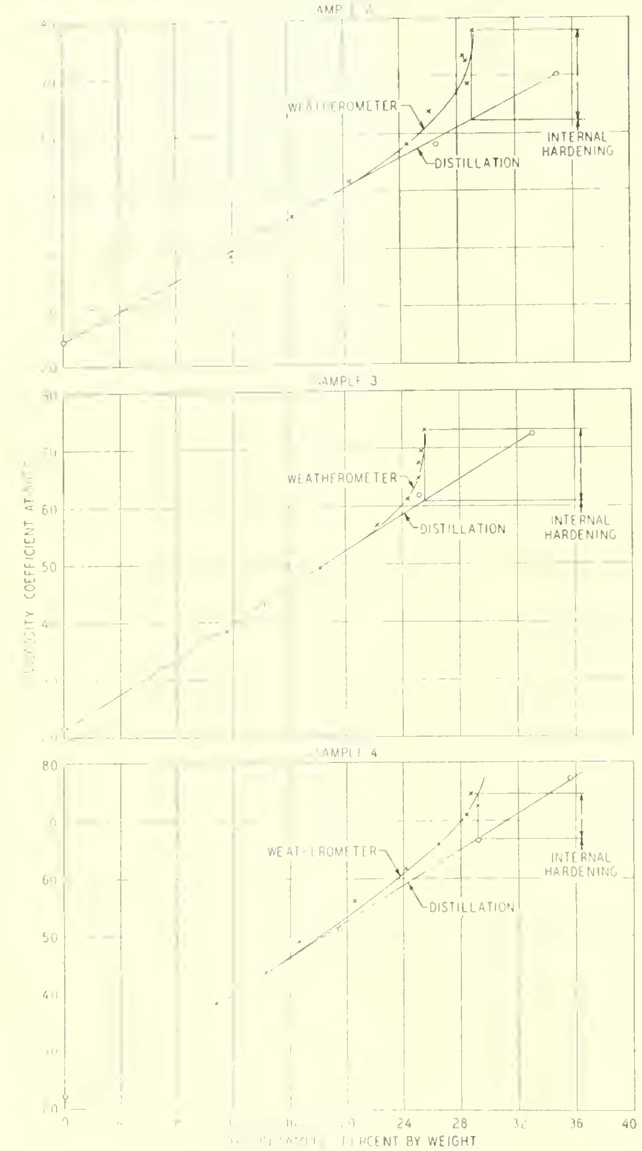

Figure 8.—Relation Between the Viscosity Coefficient of the Residues at 50° C. and the Percentage of Loss for Selected Tar Samples.

The values for the hardening index for the weatherometer residues show an increase for the longer periods of exposure, which indicates that some of the hardening has been caused by other factors in addition to evaporation. However, in order to obtain a better indication of what occurs during the different methods of hardening, the viscosity coefficients at 50° C., which are the double logarithms of the kinematic viscosities in centistokes multiplied by 100, were plotted against the percentage of loss for each residue obtained in both the weatherometer and distillation tests. These curves are shown in figure 8. It is seen from the figure that generally the values obtained for the lower amounts of loss in either the weatherometer or distillation test showed good agreement. However, the values for the distillation residues continued on a straight line while for the longer periods of exposure in the weatherometer the viscosity of the residues continued to increase with very little or no additional loss in weight, and thus the curves for the weatherometer data bend upward sharply.

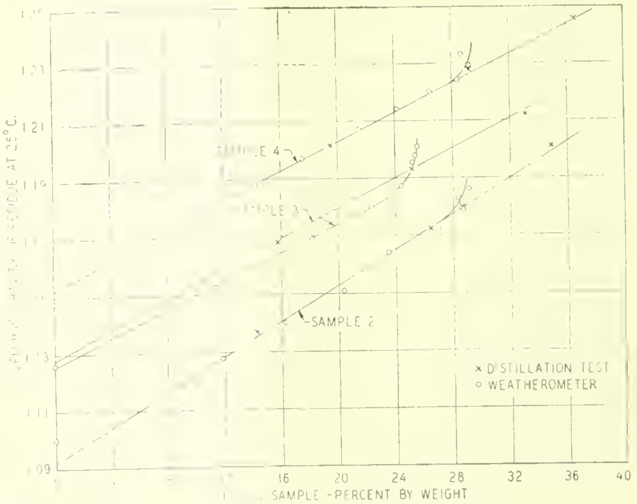

Figure 9.—Variation of Specific Gravity With Percentage of Loss for Special Tar Samples.

This difference between the values of the viscosity coefficients for the weatherometer data and those for the distillation data at equal amounts of loss may be considered a measure of the amount of internal hardening which has taken place. The relative magnitude of this difference is shown on the right-hand side of each graph in figure 8 and it is seen that for these samples the internal hardening is in the order of the amount of sulfonation residue. Sample 2, having high sulfonation indexes, shows about two times as much internal hardening as sample 4, which had very low sulfonation indexes, and sample 3 shows about 1.6 times as much as sample 4.

Figure 9 shows the specific gravities of the various residues plotted against the percentage of loss and the curves show the same trend as do those in figure 8. The points were found to be close to a straight line for the lower amounts of weatherometer loss and for all the distillation points, but the points for the longer periods of exposure showed an increase of specific gravity with very little additional loss. The agreement between the points obtained on the residues from the weatherometer and those obtained on the residues from the distillation test was good for samples 2 and 4, but for sample 3 the points from the distillation were above those for the weatherometer.

The results of the tests on the sand-tar mixes are shown in table 18. The sulfonation indexes and the amount of distillate in the tar up to 300° C. are also repeated here to facilitate comparisons. It is seen that the stability increases very rapidly at first, this increase continuing up to 200 hours and the 400-hour residues show little further increase over the 200-hour specimens. The impact tests proved to be of little value since there was little difference in any of the results, but in each case the value after 400 hours of exposure was equal to or less than the value after 200 hours, which indicates that the samples were becoming more brittle with increased weathering.

It is also interesting to note that the percentage of tar loss from these specimens after 400 hours of weathering was very nearly equal to the percentage of loss in the distillation test up to 300° C. This was not true of the ⅛-inch film exposures of the tar itself. There is on direct relationship between the sulfonation index of either portion of the distillate of the tar binder and the stability of the sand mixes. However, the data shown in table 18 indicate that the stabilities developed in the

(*Continued on p. 164*)

TRAFFIC GROWTH AND COMPOSITION 1937-1941

BY THE DIVISION OF HIGHWAY TRANSPORT, PUBLIC ROADS ADMINISTRATION

Reported by L. E. PEABODY, Senior Highway Economist

PERMANENTLY installed automatic traffic counters on the rural highways of 47 States were in operation at 657 locations in March 1942. Many of these stations have been operated for 24 hours a day every day since January 1, 1937.[1] They furnish the most complete and accurate data available for the study of traffic characteristics and trends and, from these data, many valuable conclusions may be drawn. Most of these stations are located on State highways; about 10 percent are on local roads.

At many of these points it is possible to separate the total traffic (read from the machines) into its components of passenger cars, various sizes and types of trucks, and busses as a result of manual classifications of traffic made at frequent intervals throughout each of the years and usually taken on weekdays. All of this material has been brought together for analysis.

Traffic on State highways, measured at such stations, increased 19.3 percent from 1939 to 1941, inclusive. Traffic of local passenger cars[2] increased much more (20.9 percent) than did traffic of foreign passenger cars (14.3 percent). Traffic of trucks and busses upon State highways increased much more (31.9 percent) than did total traffic.

The detailed composition of traffic at all stations where traffic could be separated by type of vehicle is listed in table 1.

Table 1.—*Composition of traffic on State highways, 1939-1941*

Type of vehicle	Percentage of total traffic		
	1939	1940	1911
Passenger cars	77.9	77.3	76.1
Light trucks	11.8	12.7	12.1
Medium trucks	3.8	4.0	4.1
Heavy trucks	1.0	.8	1.2
Tractor-truck semitrailer	4.2	4.0	5.1
Truck with full trailer	.4	.4	.5
Busses	.9	.8	.9
Total	100.0	100.0	100.0
Number of stations	275	292	200

TRAFFIC VOLUME DECLINED SHARPLY IN 1942

The increase in proportion of single-unit trucks was from 16.6 percent in 1939 to 17.4 percent in 1941. Tractor-truck semitrailers increased from 4.2 percent of the total in 1939 to 5.1 percent in 1941. Coupled with the increase in total traffic, this means an increase of 45 percent in the average number per day of tractor-truck semitrailers using the State highways during the period from 1939 to 1941, inclusive.

Figure 1 presents traffic data by months at all stations located on State highways for the period January 1939 to October 1942, inclusive, together with the same data adjusted for seasonal variation, and the trend of traffic from January 1939, to January 1942, inclusive. Adjustment for seasonal variation includes the following steps: The seasonal variation is determined for the 3 years 1939, 1940, and 1941, with the normal seasonal variation as the weighted average of these data, by months. The adjusted traffic average, presented as a broken line in figure 1, results from dividing the traffic data for each month by the corresponding monthly index of the normal seasonal variation.

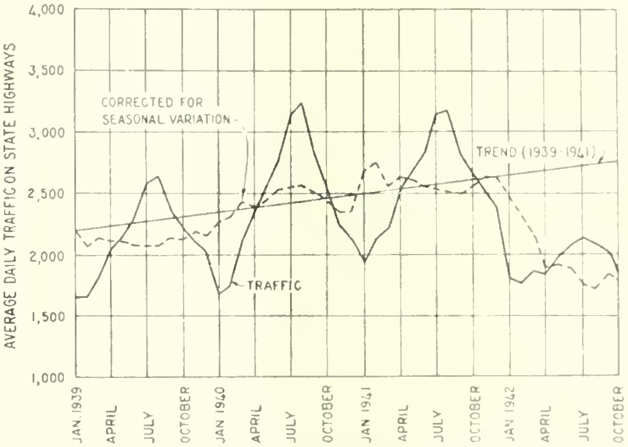

Figure 1.—Average Daily Traffic by Months at Automatic Traffic Recorders on State Highways; Average Daily Traffic Corrected for Seasonal Variation; and Trend of Traffic (1939-41).

When the adjusted traffic average by months is compared with the monthly trend values, it is found that the ratio varies from a low of 91.1 to a high of 110.4 percent during 1939, 1940, and 1941. In February 1942, the ratio first broke out of this range and dropped to 86.6 percent. For subsequent months in 1942 the ratios have been: March, 81.2; April, 70.2; May, 71.7; June, 69.3; July, 64.1; August, 60.1; September, 65.6; and preliminary data for October, 63.1 percent. Thus traffic volume in the latest month available indicates a decline of 36.9 percent from that normally to be expected. These data furnish a more realistic measure of traffic decline than the simple comparison of traffic volume by months with the volume for the same month a year previous.

One thesis of traffic growth that is sometimes presented maintains that percentage rates of growth are greatest at points of highest traffic volume; i. e., that high rates of growth are associated with large traffic volumes. Figure 2 shows the results of an examination of this thesis. All stations near, or in, defense areas were eliminated from this comparison so that no such possible disturbing element could influence the conclusions. When it is recalled that these data represent traffic counted for 24 hours per day during 365 days per year during 1937 to 1941, inclusive, at points scattered

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[1] Applications of Automatic Traffic Recorder Data in Highway Planning, by L. E. Peabody and O. K. Normann. PUBLIC ROADS, January 1941.

[2] Local vehicles are those carrying license plates of the State in which the station is located.

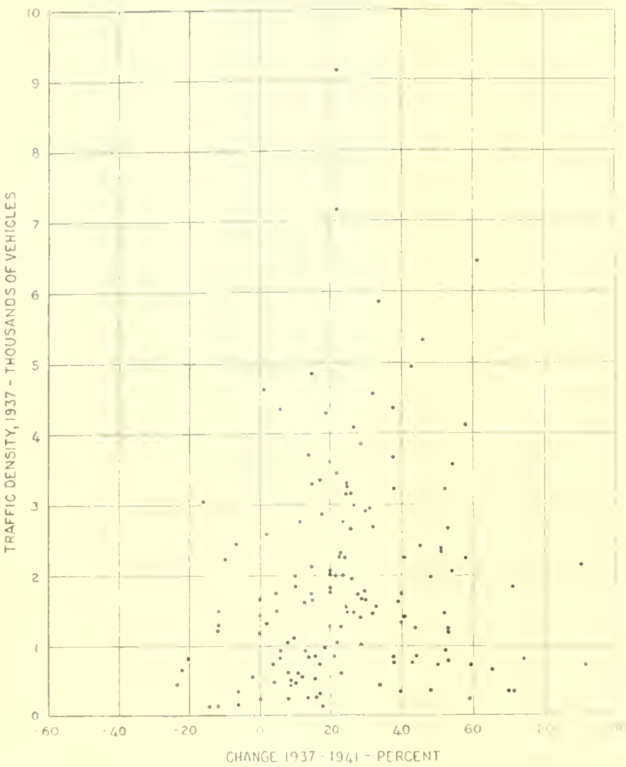

Figure 2.—Traffic Volume and Percentage of Change in Traffic Volume, 1937–41. (Each Dot Is One Station.)

all over the United States, the indefensibility of this thesis seems evident. From figure 2, it is seen that at traffic volumes of less than 1,000 vehicles per day, amounts of change vary from an increase of more than 90 percent to a decrease of more than 20 percent. For traffic volumes of from 4,000 to 5,000 per day, the amounts of increase vary from 1 percent to 58 percent.

COMPARISON MADE OF VOLUME CHANGES ON ROUTES CARRYING DIFFERENT TYPES OF TRAFFIC

One hundred and twenty-seven State highway routes could be classified by the State highway departments according to the dominant purposes of the travelers who used them as: Tourist, industrial, farm-to-market traffic, etc. In table 2 a comparison is made of the amount of traffic increase by general type of use during 1937 to 1941, inclusive, and from 1937 to 1940, inclusive. It must be realized that such route classifications according to use are not, and cannot be, truly representative of all the traffic using the route. There will inevitably be some mixture in purpose of use upon all, except possibly a few lightly traveled, routes. These classifications are based upon the purposes of the majority and, in many instances, nearly all of the users.

When the rates of growth in the period from 1937–41 are compared with those in the period 1937–40, some interesting conclusions may be drawn. One is that, at least for this group of 127 routes, the increase in 1941 exceeded the total increase during the three previous years. Another conclusion comes from a comparison of the amounts of increase by purpose of use. When data for 1941 are included the tourist routes show an increase of 38.5 percent as against 21.7 percent for farm-to-market or agricultural routes. If the 1941 traffic is excluded, the increase on farm-to-market routes (15.4 percent) exceeds that upon tourist routes (14.7 percent). In other words, judging from traffic increases upon all routes subject to reasonably accurate classification according to purpose of use, it would seem that tourists took a sizeable "last fling" in 1941. This may be considered together with the far above normal sales of passenger cars in 1941, and with data on the number of automobiles entering national parks during 1941 as compared with those in previous years.

Table 2. *Increase in traffic during 1937–41 and 1937–40 according to type of traffic moving over the route*

Type of traffic	Number of routes	Average density, 1937	Percent increase	
			1937–41	1937–40
Tourist	18	1,953	38.5	14.7
Mines	6	1,020	37.7	14.7
Industrial	8	2,102	31.3	7.2
Metropolitan	12	3,956	28.8	11.6
Tourist and farm-to-market	26	2,041	24.6	12.3
Tourist and commercial	20	1,701	22.4	12.6
Farm-to-market or agricultural	37	1,182	21.7	15.4
Total or average	127	1,861	27.5	12.8

Also significant is the fact that routes used mainly by traffic to mines showed almost as great an increase in 1941 as did those used mainly by tourists. However, it should be noted that only six routes to mines were included.

The proportion of traffic carried during various periods of the day has shown almost no change during the years 1937 to 1941 inclusive. Data at 10 stations, with one each in Pennsylvania, Michigan, South Carolina, Florida, Colorado, Washington, and two each in California and Texas, were used to develop this conclusion. The data are presented in table 3.

Table 3.—*Proportion of total traffic during designated periods for the month of July; data for 1937 to 1941, inclusive*

	6 a. m. to 6 p. m.	6 p. m. to 10 p. m.	10 p. m. to 6 a. m.
	Percent	*Percent*	*Percent*
1937	64.7	21.2	14.1
1938	65.4	21.2	13.4
1939	65.8	21.3	12.9
1940	64.9	21.3	13.8
1941	63.9	21.2	14.9

MONTHLY VARIATIONS IN TRAFFIC REMAIN STABLE FROM YEAR TO YEAR

Changes in seasonal variation in traffic have also been small at the automatic traffic recorders during the period from 1939 to 1941, inclusive, as indicated in figure 3.

The seasonal variation is derived by dividing each month's traffic by the traffic in the average month of the year. In this manner the monthly variations in several years may be compared, even though there may be considerable difference in the total traffic of these years. Figure 3 indicates, not that there have been no changes in traffic volume from 1939 to 1941 (actually it is known that traffic increased nearly 20 percent in this period), but that the monthly variations in traffic remained quite stable from year to year.

It is interesting to compare the seasonal variations in motor-vehicle traffic and railway passenger traffic (commutation traffic excluded) during the year 1939. This is presented in figure 4. When it is understood

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DESIGN OF SLAB AND STRINGER HIGHWAY BRIDGES

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A report of an investigation conducted by the University of Illinois Engineering Experiment Station, the Public Roads Administration, and the Illinois Division of Highways

THE slab and stringer bridge is one of the simplest and most conveniently constructed types of highway bridges. Three kinds of such bridges are considered in this paper:

1. The I-beam bridge, which consists of a concrete slab continuous over steel beams, with the beams in the direction of traffic;
2. The composite I-beam bridge, which has shear developers or shear connectors between the beams and the slab which make the structure act as a composite T-beam; and
3. The concrete T-beam bridge, with concrete beams running in the direction of traffic.

The terminology used in describing the component parts of these bridges is illustrated by figures 1 and 2, which are typical views of an I-beam bridge and a concrete T-beam bridge, respectively. The span of the bridge is denoted by a, the spacing of the beams by b, and the thickness of the slab by h. The main reinforcement in the slab is in the direction transverse to the beams, and will be called the transverse reinforcement; the secondary reinforcement in the slab is in the direction of traffic, and will be called the longitudinal reinforcement.

Analyses of slab and stringer bridges are available in Bulletin No. 336 of the University of Illinois Engineering Experiment Station.[1] The bulletin contains coefficients for moments in the beams and the slab for bridges of various proportions. The moment coefficients were determined by analyses taking into account the deflections of the supporting beams. Empirical relations for maximum moments due to standard truck loadings were determined, and are used as a basis for the present studies.

In this paper the comparatively simple formulas which are given for the moments controlling the design of slab and stringer bridges are applicable to a wide range of sizes and proportions of bridges. The procedures described herein are applicable generally to structures with span lengths from 20 to 80 feet, and beam spacings from 5 to 8 feet, for both H–15 and H–20 standard truck loads and truck train loads. The working stresses considered are 18,000 pounds per square inch for both reinforcing and structural steel, and 1,000 pounds per square inch for concrete. The modulus of elasticity of concrete is taken as 3,000,000 pounds per square inch. These values are in accord with the requirements of the current Standard Specifications for Highway Bridges, of the American Association of State Highway Officials, Third Edition, 1941. For other conditions and working stresses, one may derive approximate formulas of the same type as those given herein from the original equations given in Bulletin No. 336.

FUNDAMENTAL RELATIONS OUTLINED

The design of a simple span slab and stringer bridge, after the span and beam spacing have been chosen, is primarily controlled by the maximum moment in the slab at the center of the panel and by the maximum moment in the beams. The former determines the thickness of the slab and the latter the cross-section of the beams. Both of these moments are dependent on the relative stiffness of the beams and the slab. The relative stiffness, a dimensionless quantity denoted by the symbol H, is defined by the relation

$$H = \frac{E_b I_b}{a\,EI} \qquad (1)$$

where E_b and E are the moduli of elasticity of the materials in the beams and in the slab respectively, in units of lb. per sq. in.,
I_b is the moment of inertia of the cross-section of the beam, in units of in.^{4}
I is the moment of inertia per unit of width of the cross-section of the slab in units of in.^{4} per in. In general, I is taken as $\frac{h^3}{12}$ where h is the thickness of the slab.
a is the span of the bridge, in inches.

The effect of H upon the moments controlling the design is small. This is fortunate, since the magnitude of H is subject to some uncertainty. It is reasonable to use as a modulus of elasticity of the concrete the modulus applying to short time loading, since the major effect of H is on the live load moments. The depth of the slab will have a large influence on the value of H. For the three types of bridges considered in this paper, and for the working stresses used herein, the range in values of H can be summarized reasonably well for designs ordinarily encountered, and the formulas for moments can thereby be made independent of H.

The standard truck loading for which the calculations are made is that specified as the H truck loading in the Standard Specifications for Highway Bridges, previously cited. The standard H truck has the wheels of each axle spaced 6 feet apart, with front and rear axles spaced 14 feet apart. Each of the rear wheels carries a weight which is four-tenths of the total weight of the truck, and each of the front wheels carries a weight of one-tenth the total weight of the truck, or one-fourth the rear wheel weight. The weight of the truck in tons is designated by a numeral following H, as H–20. The rear wheel load P, in terms of which moment coefficients are stated, is the weight on a rear wheel in pounds increased by the impact factor.

Each truck is considered to occupy the central part of a 10-foot traffic lane, and therefore the minimum distance between the center of a wheel and the face of a curb is taken as 2 feet; the minimum distance between centers of wheels of trucks in adjacent lanes is taken as 4 feet. The methods for design given in this paper consider additional trucks in a lane by use of a truck train loading[2] in which the standard truck may be

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[1] See notice on p. 166.

[2] The truck train loadings are used herein rather than uniform lane loadings since the former are more representative of actual loadings, and since, in general, the maximum moments in the beams due to lane loadings distributed uniformly are smaller than those due to wheel loads distributed in accordance with the equations given in this paper.

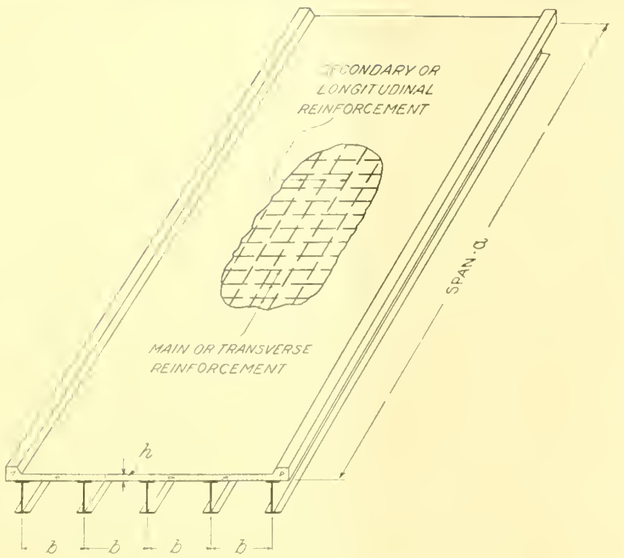


FIGURE 1.—TYPICAL I-BEAM BRIDGE.

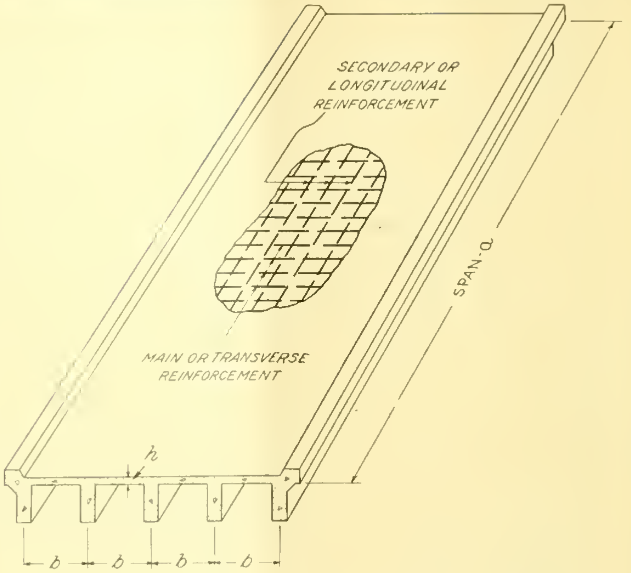


FIGURE 2.—TYPICAL CONCRETE T-BEAM BRIDGE.

followed or preceded at a distance of 30 feet by a similar truck weighing three-fourths as much.

The maximum positive live load moment at the center of a panel of the slab due to standard truck loading is the transverse moment M_t and is given by equation 12 of Bulletin No. 336, as follows:

$$M_t=P\left[\frac{1.16}{3+10\frac{c}{b}}-0.040+0.024\frac{a}{10\text{ feet }\sqrt{H}}\right.$$
$$\left.+0.010\frac{a-28\text{ feet}}{10\text{ feet }\sqrt{H}}\right].$$

In this equation, c is the equivalent diameter of the circular area over which the wheel load is assumed to be distributed, ordinarily taken as 1.25 feet. The quantities a and b are measured in feet and P is measured in pounds. Consequently, the moment will be in units of foot-pounds per foot of width. The last term in the equation takes account of the front wheel loads of the truck, and is to be dropped for spans of less than 28 feet.

The maximum live load moment in a beam, as given by equations 24 and 25 of Bulletin No. 336, is

$$M_b=k\left[\frac{1}{4}Pa+\frac{1}{16}P(a-28\text{ feet})\right]$$

$$k=\frac{b\left(1-\frac{b}{aH}\right)}{4.40\text{ feet}+0.42\frac{a}{10\sqrt{H}}}$$

where the last term in the first equation takes account of the front wheel loads and is to be dropped for spans of less than 28 feet. For spans greater than 60 feet, the moment due to an additional truck in each lane may be approximated by the added term

$$k\left[\frac{1}{20}P(a-60\text{ feet})\right]$$

In these expressions, a and b are measured in feet, P is measured in pounds, and the moments in the beam will be in units of foot-pounds.

It is shown in Bulletin No. 336 that the dead load moments in the slab are affected by the method of construction and are subject to considerable uncertainty unless the construction procedure is very carefully controlled. The moment coefficients for dead load given here are intended to apply to average conditions and are generally on the safe side to allow for some of the uncertainties. For **I**-beam bridges, whether composite action is provided or not, the dead load moment in the slab at centers of panels may be taken as $\frac{1}{8}\,wb^2$ where w is the magnitude of the dead load per unit of area measured in pounds per square foot. For concrete T-beam bridges, this moment may be taken as $\frac{1}{10}\,wb^2$ because of the greater stiffness of the concrete T-beams. The live and dead load moments at other points in the slab may be obtained from Bulletin No. 336, but are not necessary for designs made according to the procedures given herein, whereby certain proportions of reinforcement are provided at other points.

The dead load moment in edge or interior beams may be determined from the weight of one panel of the slab, the weight of the beam, and whatever paving allowance is considered. The total dead load moment provided for in all the beams should not be less than the total statical dead load bending moment due to the entire weight of the bridge, including handrails, curbs, sidewalks, and other such details. Similarly, the total live load moment provided for should not be less than the total statical live load bending moment.

COMPOSITE ACTION OBTAINED WITH SHEAR CONNECTORS

Composite action of the slab with steel beams may be considered if adequate provision is made for shear connectors. Such shear connectors may be made up of channels, plates, angles, or other structural elements, welded or riveted to the top flange of the beam, and embedded in the concrete of the slab. To prevent uplift there should be a mechanical anchorage of the connector in the concrete. The load to be carried by the shear connector may be computed by the ordinary formulas of elementary mechanics from the shear on the composite T-beam section.

When the composite structure is constructed in the usual manner, without false work supporting the beams during construction of the slab, the dead load from both the slab and the beam is carried by the steel beam section alone, but the live load is carried by the composite T-beam section. Where the dead load shear is carried by the steel beam section alone, it should be noted that the shear connector does not have to provide for the dead load action.

The dead load shear varies linearly from the reaction at the end of a beam to zero at the center. It is recommended that the live load shear in a beam be computed from the shear for either truck or truck train loadings by the use of the same proportion, *k*, of a wheel load that is used in determining the moment in the beam. The total force acting on any shear connector may then be computed from the longitudinal shear per unit of length between the beam and the slab, multiplied by the spacing between the shear connectors.

For design purposes, the tentative recommendation is made that the distribution of load over the face of the shear connector be considered linear, from zero at the top of the connector to a maximum at the junction of the connector and the top flange of the beam. The stress in bearing on the concrete may be taken equal to the usual compressive working stress for concrete, and the flexural stress in the shear connector may be taken as the usual working stress for the steel. The spacing of the shear connectors should be not more than 3 to 4 times the depth of the slab. The usual criteria should cover the design of the connection between the shear connector and the beam.

A limited number of tests indicate that when the shear connector is a channel having one flange attached to the beam, the above working stresses may safely be doubled, or the load the shear connector must provide for may be reduced by one-half. Tests of quarter-scale models of I-beam bridges with such shear connectors indicated that the strength of the slab and of the beams may be developed without any evidence of failure of the shear connectors, even with a somewhat less conservative design than that proposed. These recommendations are subject to revision in the light of tests that are now under way.

DESIGN OF SIMPLE SPAN I-BEAM BRIDGES GIVEN

For simple span I-beam bridges, designs of structures with the working stresses considered in the preceding sections indicate values of *H* falling in the range given by the equation:

$$H=(0.4\text{ to }0.8)\frac{a}{10\text{ feet}} \qquad (2)$$

where a is measured in feet.

This relation applies both for H–15 and H–20 loading, since the difference in slab thickness required for these loadings has the effect of making the values of *H* nearly the same. The effect of considering a paving allowance of 25 pounds per square foot is to give only slightly increased values of *H*.

Under ordinary conditions the slab may be designed for a live load moment M_t given by equation 3, which is independent of *H*:

$$M_t=P\left[\frac{1.16}{3+10\frac{c}{b}}+\frac{a}{1000\text{ feet}}\right] \qquad (3)$$

This is the transverse moment at the centers of the panels of the slab (in the direction perpendicular to the beams). The corresponding dead load moment in the slab may be taken as $\frac{1}{8}wb^2$. For a first approximation it will usually be adequate to assume that the dead load moment is 10 percent of the live load moment.

The moment given by equation 3 will control the amount of transverse reinforcement in the bottom of the slab at the centers of panels. For ordinary designs, the empirical equations given in Bulletin No. 336 indicate that approximately one-fourth as much steel should be used at the top of the slab for transverse reinforcement at the centers of the panels as is used in the bottom. Over the beams the transverse reinforcement at the bottom of the slab should be about one-half, and at the top of the slab about two-thirds to three-fourths, of the transverse bottom steel at the centers of the panels of the slab.

The longitudinal reinforcement required in the slab (in the direction parallel to the beams) may be nearly as much as the transverse reinforcement for short span structures but will be considerably less for long span structures. The following approximation is recommended for the ratio of longitudinal to transverse reinforcement at the bottom of the slab in the central half of the panels:

$$\frac{\text{Longitudinal reinforcement}}{\text{Transverse reinforcement}}=1-\frac{a}{200\text{ feet}} \qquad (4)$$

where a is measured in feet.

The longitudinal reinforcement in the bottom of the slab over the beams depends on a number of factors, the effects of which are difficult to estimate. However, it seems desirable to provide from one-half to one-third the amount of longitudinal reinforcement that is provided in the central half of the panel, the smaller amount being used for the longer spans.

The live load moment in the beams is given by the following approximate equation,

$$M_b=k\left[\frac{1}{4}Pa+\frac{1}{16}P(a-28\text{ feet})+\frac{1}{20}P(a-60\text{ feet})\right] \qquad (5)$$

where the last term is to be dropped for spans of less than 60 feet, and the second term is to be dropped for spans of less than 28 feet. The quantity k may be considered as the proportion of a wheel load which effectively contributes to the moment in a beam, and is given by the relation:

$$k=\frac{b}{6\text{ feet}} \qquad (6)$$

where b is the beam spacing in feet.

It will be noted that *H* does not enter into these relations, and it has been found that this approximation for the moment in the beams is reasonably accurate for designs falling within the range of proportions considered here.

The dead load moment in the interior beams is determined from the weight of one panel of the slab and the weight of the beam, plus any paving allowance that may be considered. All the beams, edge or interior, should have the same section since the maximum dead plus live load moments in all beams are very nearly the same.

TYPICAL DESIGN COMPUTATIONS GIVEN FOR SIMPLE SPAN COMPOSITE I-BEAM BRIDGE

A composite I-beam bridge will in general offer some economy over a corresponding simple I-beam structure. If composite action is provided for by adequate shear connectors, designs of structures with the working stresses considered here indicate values of *H* generally about twice as great as for an I-beam bridge of the same span and beam spacing.

Under ordinary conditions the slab may be designed for a live load moment M_t given by the following equation:

$$M_t = P\left[\frac{1.16}{3+10\frac{c}{b}} - 0.025 + \frac{a}{1000 \text{ feet}}\right] \quad (7)$$

The corresponding dead load moment may be taken as $1/8\ wb^2$. The transverse reinforcement in the slab may be distributed in the same proportions as for an I-beam bridge not designed for composite action. The relative amount of longitudinal reinforcement at the bottom of the slab in the central half of the various panels may be computed from equation 4. However, over the beams the longitudinal steel in the slab can be considerably less, or may even be omitted entirely.

The live load moment in the beams is given by equation 5 where the quantity k is determined by the relation:

$$k = \frac{b}{5.5 \text{ feet}} \quad (8)$$

The dead load moment in the beams is determined from the weight of one panel of the slab and the weight of the beam, plus any paving allowance that may be considered. Where the beams are not supported during construction of the slab, the dead load moment is carried by the steel beam alone, whereas live load moment is carried by the composite section.

The effective width of the slab, acting as the upper flange of the composite beam, may be taken as b, the spacing of the beams. However, the slab is in addition subjected to local longitudinal flexure in the neighborhood of concentrated loads. This flexure causes essentially a local stress. Furthermore, the longitudinal action of the slab in local flexure is not as significant as the transverse action. Therefore, it is recommended that the compressive stress in the slab, acting as the upper flange of the composite beam, be limited to the usual working stress in concrete, neglecting the local longitudinal flexure. However, in order to provide for extreme cases, it is suggested that the sum of the actual compressive stress due to the action as the upper flange of the beam, plus 50 percent of the stress due to local longitudinal flexure, be limited to an amount one-third greater than the usual working stress for concrete in compression. For this purpose, the ratio of the compressive stress in the slab due to local longitudinal flexure, to the compressive stress due to transverse flexure, may be considered the same as the ratio of longitudinal reinforcement to transverse reinforcement as given in equation 4.

To illustrate the design procedure recommended herein, typical design computations are given in figures 3, 4, and 5. A complete design is given of a composite I-beam bridge of 56-foot 9-inch span, and 6-foot 6-inch beam spacing, corresponding to a 26-foot roadway, for H 15 loading. The calculations are self-explanatory.

In general, a more efficient steel section for composite action would be a beam having a heavier lower flange than upper flange. Such a section may be built up by riveting or welding a cover plate to the lower flange of a rolled beam.

DESIGN OF SIMPLE-SPAN BRIDGES WITH CONCRETE T-BEAM DESCRIBED

In a concrete T-beam bridge the moment of inertia of the supporting beam will usually be very large. The values of H computed for representative designs are from 4 to 8 times the values given by equation 2. The following rules for design are based on a structure in which all the beams have the same section below the slab.

For usual conditions the slab may be designed for a live load moment M_t given by the following equation:

$$M_t = P\left[\frac{1.16}{3+10\frac{c}{b}} - 0.025 + \frac{a}{2000 \text{ feet}}\right] \quad (9)$$

The corresponding dead load moment in the slab may be taken as $\frac{1}{10}\ wb^2$.

The moment given by equation 9 will control the amount of transverse reinforcement in the bottom of the slab at centers of panels. For ordinary designs approximately one-fourth as much steel should be used at the top in the transverse direction at the centers of panels of the slab as is used in the bottom. Over the beams the transverse reinforcement at the bottom of the slab should be about one-fourth to one-third, and at the top of the slab about three-fourths to seven-eighths of the transverse steel at the center of the panel, at the bottom of the slab.

The longitudinal reinforcement in the slab (in the direction of the beams) will be considerably less than the transverse reinforcement. The following approximation is recommended for the ratio of longitudinal to transverse reinforcement at the bottom of the slab in the central half of panels:

$$\frac{\text{Longitudinal reinforcement}}{\text{Transverse reinforcement}} = 0.75 - \frac{a}{400 \text{ feet}} \quad (10)$$

Over the beams the longitudinal steel may be a purely nominal amount or may even be omitted entirely.

In the concrete T-beam bridge, the T-beam section carries the beam moment for both live and dead loads. The dead load moment in the beams is determined from the weight of one panel of the slab and the weight of the beam. The live load moment is given by equation 5 in which the value of k is as follows:

$$k = \frac{b}{5 \text{ feet}} \quad (11)$$

The compressive stress in the concrete in the longitudinal direction for the T-beam bridge should be subject to the same limitations previously discussed for the composite I-beam bridge, with the exception that the ratio of the compressive stress in the slab due to local longitudinal flexure, to the compressive stress due to transverse flexure, is the same as the ratio of longitudinal reinforcement to the transverse reinforcement as given in equation 10.

For the concrete T-beam structure, the design of stirrups in the beam may be made with the assumptions regarding vertical shear in the beams that were previously discussed in the section concerning design of shear connectors for composite I-beam bridges.

BASES FOR DESIGN OF CONTINUOUS SLAB AND STRINGER BRIDGES AND SKEW BRIDGES GIVEN

No analyses of continuous slab and stringer bridges are available. It is felt, however, that the magnitudes of the maximum moments in continuous bridges may be approximately determined from the relations given above for simple span bridges. The following princi-

SPAN OF BEAMS, a = 56'-9"
BEAM SPACING, b = 6'-6"

H-15 TRUCK LOADING
DIAMETER OF LOADED AREA, c = 1'-3"

A.A.S.H.O. SPECIFICATIONS (1941)

f_s = 18,000 LB. PER SQ. IN.
f_c = 1,000 LB. PER SQ. IN.
n = 10

	BEAMS	SLAB
WEIGHT ONE REAR WHEEL =	12,000 LB.	12,000 LB.
IMPACT =	3,300	3,600
DESIGN LOAD, P =	15,300 LB.	15,600 LB.

DESIGN OF SLAB

FROM EQUATION 7, $$M_t = 15600\left[\frac{1.16}{3+\left(10 \times \frac{1.25}{6.5}\right)} - 0.025 + \frac{56.75}{1000}\right] = 4,170 \text{ FT. LB. PER FT.}$$

DEAD LOAD MOMENT FOR $6\frac{3}{4}$" SLAB = $\frac{1}{8} wb^2 = \frac{1}{8} \times 84.4 \times 6.5^2 =$ 445

TOTAL MOMENT = 4,615 FT. LB. PER FT.

REQUIRED EFFECTIVE DEPTH = $5\frac{1}{2}$"
1" COVER + $\frac{1}{2}$ BAR DIAMETER = $1\frac{1}{4}$"
TOTAL DEPTH = $6\frac{3}{4}$"

TRANSVERSE BOTTOM STEEL AT CENTERS OF PANELS

$$A_s = \frac{4615 \times 12}{18000 \times 0.885 \times 5.5} = 0.632 \text{ SQ. IN. PER FT.}$$

USE: $\frac{1}{2}$" ϕ AT $3\frac{3}{4}$" C.C.

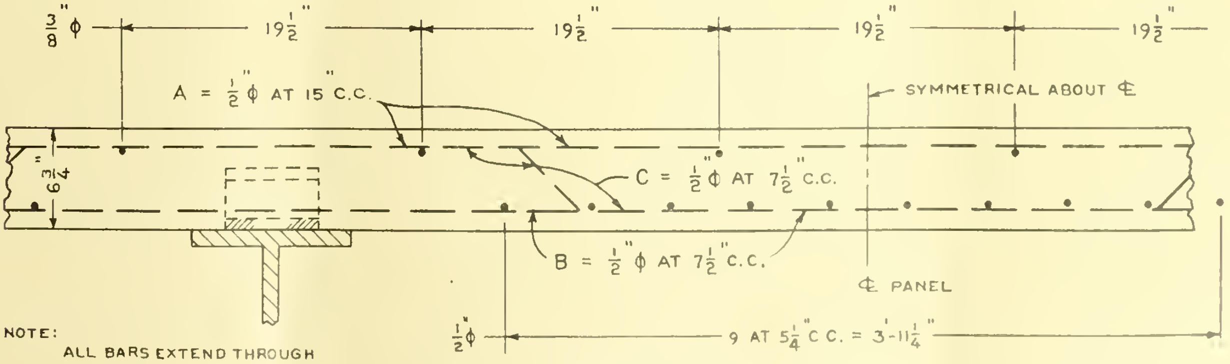
LONGITUDINAL BOTTOM STEEL AT CENTERS OF PANELS

FROM EQUATION 4

$$A_s = \left(1 - \frac{56.75}{200}\right) 0.632 = 0.453 \text{ SQ. IN. PER FT.}$$

USE: $\frac{1}{2}$" ϕ AT $5\frac{1}{4}$" C.C.

REINFORCING BAR LAYOUT



CROSS SECTION OF SLAB

Figure 3.—Illustrative Design of Composite I-Beam Bridge—Design of Slab.

DESIGN OF BEAMS

FROM EQUATION 8 $k = \frac{6.5}{5.5} = 1.182$

FROM EQUATION 5 LIVE LOAD $M_b = 1.182 \left[\frac{1}{4} \times 15300 \times 56.75 + \frac{1}{16} \times 15300 (56.75 - 28) \right]$

$= 289{,}000$ FT. LB. $= 3{,}470$ IN. KIPS

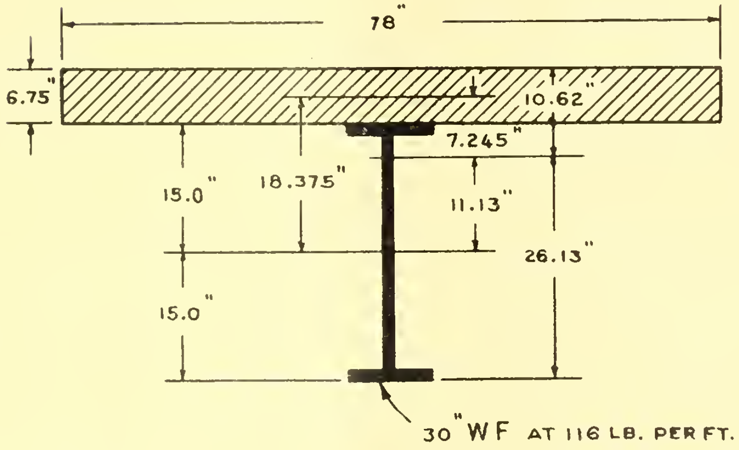
DEAD LOAD: WEIGHT OF SLAB $= 0.0844 \times 6.5 = 0.549$ KIPS PER FT.

WEIGHT OF BEAM, (ASSUMED) $= 0.116$

TOTAL DEAD LOAD $= 0.665$ KIPS PER FT.

DEAD LOAD MOMENT $= \frac{1}{8} \times 0.665 \times (56.75)^2 \times 12 = 3{,}215$ IN. KIPS

(CARRIED BY I-BEAM ALONE)



A	y	Ay	Ay^2 AND I_o
$\frac{78 \times 6.75}{10} = 52.60$	18.375	966	17,770
			200
34.13	0	0	4,919
86.73		966	22,889
	$\bar{y} = 11.13$		
	$A\bar{y}^2 =$		$-10{,}770$

$I = 12{,}119$ IN.^{4}

$\frac{I}{c}$ FOR I-BEAM ALONE $= 327.9$ IN.^{3}

STEEL STRESS

$$f_s = \left(\frac{3215}{327.9} + 3470 \times \frac{26.13}{12119} \right) 1000 = 9800 + 7480 = 17{,}280 \text{ LB. PER SQ. IN.}$$

CONCRETE STRESS

$$f_c = \left(3470 \times \frac{10.62}{12119} \right) \frac{1000}{10} = 304 \text{ LB. PER SQ. IN.}$$

Figure 4.—Illustrative Design of Composite I-Beam Bridge—Design of Beams.

DESIGN OF SHEAR CONNECTORS

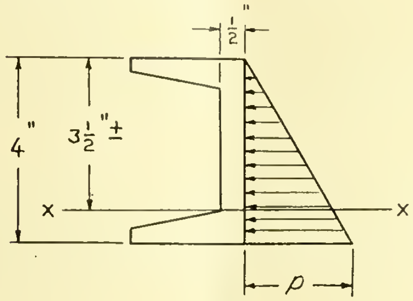
TAKE VERTICAL SHEAR, V = 24,500 LB. AT ENDS OF BRIDGE,

= 15,800 LB. AT QUARTER POINTS,

= 10,200 LB. AT MIDSPAN.

$$\text{HORIZONTAL SHEAR} = q = \frac{VyA'}{I} = V \times \frac{7.245 \times 52.6}{12119} = 0.0314\,V$$

FOR SHEAR CONNECTORS: USE 4" × 13.8 LB. CAR CHANNELS 8" LONG



NOTE THAT ALLOWABLE LOAD ON CONNECTOR COMPUTED FROM USUAL ALLOWABLE STRESSES MAY BE DOUBLED BECAUSE CHANNEL IS USED.

$$\text{ALLOWABLE LOAD} = 2\left(\tfrac{1}{2} \times p \times 4 \times 8\right) = 32\,p$$

IF CONCRETE STRESS GOVERNS: $p = f_c$ = 1,000 LB. PER SQ. IN.

ALLOWABLE LOAD = 32 × 1000 = 32,000 LB.

IF STEEL STRESS GOVERNS:

$$\text{MOMENT ON LINE X-X} = \frac{3.5}{4.0}\,p \times \frac{3.5}{2} \times \frac{3.5}{3} \times 8 = 14.29\,p$$

$$\frac{I}{C} \text{ OF CHANNEL WEB} = \frac{bd^2}{6} = \frac{8 \times 0.5^2}{6} = 0.333 \text{ IN.}^3$$

$$f_s = 18000 = \frac{m_x C}{I} = \frac{14.29}{0.333}\,p = 42.91\,p \qquad p = \frac{18000}{42.91} = 419 \text{ LB. PER SQ. IN.}$$

ALLOWABLE LOAD = 32 × 419 = 13,410 LB.

$\therefore$ STEEL STRESS GOVERNS.

SPACING OF CONNECTORS

AT ENDS: $q = 0.0314 \times 24{,}500$ = 770 LB PER IN., $\frac{13{,}410}{770}$ = 17.4 IN. SPACING

AT $\frac{1}{4}$ POINTS: $q = 0.0314 \times 15{,}800$ = 496 LB. PER IN., $\frac{13{,}410}{496}$ = 27.0 IN. SPACING

AT MIDSPAN: $q = 0.0314 \times 10{,}200$ = 320 LB. PER IN., $\frac{13{,}410}{320}$ = 41.9 IN. SPACING

MAXIMUM SPACING: 4 X DEPTH OF SLAB = 4 X 6.75 = 27 IN. SPACING

SHEAR CONNECTOR SPACING DIAGRAM

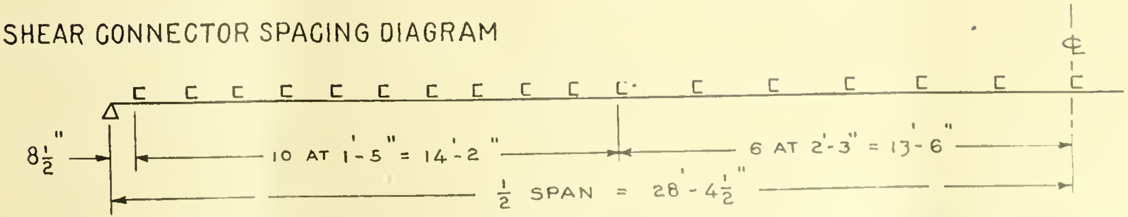


FIGURE 5.—ILLUSTRATIVE DESIGN OF COMPOSITE I-BEAM BRIDGE—DESIGN OF SHEAR CONNECTORS.

ples seem to offer a reasonable basis for the design of a continuous bridge:

1. The slab may be designed as for a simple-span bridge having a span of the beams, *a*, equal to the distance between the "points of inflection" for the particular span of the continuous bridge.

2. The proportion of a single wheel load to be used in determining moments in the beams may be determined from equations 6, 8, or 11, for bridges with steel beams, composite I-beams, or concrete beams, respectively. The moments due to a wheel load should be determined in the usual way for a continuous beam.

3. For I-beam bridges with no composite action provided for, longitudinal reinforcement for negative moments in the slab should be provided in the region where the beam moments are negative. For composite I-beam bridges or for concrete T-beam bridges the whole slab will be in tension, and may require additional reinforcement accordingly, in the regions of negative beam moments.

No analyses of skew bridges have been made and, in general, the formulas for moments given herein are based only on studies of right bridges. In the absence of other information, however, it is suggested that a skew bridge may be designed as a right bridge having the same length of beams.

(*Continued from p. 154*)

laboratory tests are more affected by the unsulfonated residue in the portion of the distillate between 300° and 355° C. than by the unsulfonated residue in the distillate up to 300° C. and generally the mixes made with tars having high sulfonation indexes for the distillate between 300° C. and 355° C. developed lower final stabilities than did those with low sulfonation indexes for this fraction.

TABLE 18.—*Results of tests on sand-tar mixes*

Sample No.	Time exposed	Sulfonation index		Weight of distillate to 300° C.	Loss of weight of tar		Toughness at 25° C.	Hubbard-Field stability at 25° C.
		Distillate to 300° C.	Distillate from 300° to 355° C.		In 2-inch cylinder	In 1-inch cylinder		
	Hours			*Percent*	*Percent*	*Percent*		*Pounds*
1	0	6.7	2.2	34.6	0	0	-----	80
	60	-----	-----	-----	24	31	3	1,550
	200	-----	-----	-----	32	39	3	3,750
	400	-----	-----	-----	35	43	2	3,840
	0	8.9	2.8	33.3	0	0	-----	190
	60	-----	-----	-----	23	27	4	1,825
	200	-----	-----	-----	30	36	3	3,750
	400	-----	-----	-----	33	41	2	4,050
3	0	3.8	1.4	31.5	0	0	-----	170
	60	-----	-----	-----	23	28	4	2,145
	200	-----	-----	-----	29	35	4	5,050
	400	-----	-----	-----	31	39	3	5,015
4	0	.7	(1)	34.6	0	0	-----	185
	60	-----	-----	-----	25	33	3	4,415
	200	-----	-----	-----	32	35	3	8,200
	400	-----	-----	-----	34	37	2	8,200
6	0	4.4	.3	26.5	0	0	-----	165
	60	-----	-----	-----	21	27	5	3,345
	200	-----	-----	-----	25	26	5	7,000
	400	-----	-----	-----	28	29	3	8,000
7	0	1.6	.7	30.2	0	0	-----	175
	60	-----	-----	-----	22	29	5	2,210
	200	-----	-----	-----	27	26	5	5,400
	400	-----	-----	-----	29	35	3	5,450
8	0	10.4	2.5	29.4	0	0	-----	100
	60	-----	-----	-----	18	27	3	1,810
	200	-----	-----	-----	28	33	3	3,150
	400	-----	-----	-----	33	36	2	4,050
9	0	9.8	.3	27.9	0	0	-----	150
	60	-----	-----	-----	22	27	4	3,850
	200	-----	-----	-----	29	33	3	7,650
	400	-----	-----	-----	31	35	2	8,350
10	0	(1)	0	27.9	0	0	-----	225
	60	-----	-----	-----	19	22	4	4,730
	200	-----	-----	-----	25	29	4	9,000
	400	-----	-----	-----	26	38	2	9,700
11	0	.4	.06	17.5	0	0	-----	580
	60	-----	-----	-----	11	6	6	5,250
	200	-----	-----	-----	17	16	2	11,400
	400	-----	-----	-----	18	20	2	11,700
12	0	4.9	2.0	31.0	0	0	-----	150
	60	-----	-----	-----	20	18	1	1,360
	200	-----	-----	-----	30	21	2	4,850
	400	-----	-----	-----	32	30	2	4,850
13	0	3.9	1.6	25.3	0	0	-----	190
	60	-----	-----	-----	16	21	4	2,000
	200	-----	-----	-----	25	25	3	5,600
	400	-----	-----	-----	26	26	2	6,150
14	0	3.6	.9	33.8	0	0	-----	100
	60	-----	-----	-----	25	26	3	1,450
	200	-----	-----	-----	37	35	3	5,600
	400	-----	-----	-----	38	39	2	5,600
15	0	3.3	1.4	32.6	0	0	-----	160
	60	-----	-----	-----	22	25	4	2,650
	200	-----	-----	-----	30	26	3	5,400
	400	-----	-----	-----	30	28	2	5,650

[1] Trace.

This conclusion may seem at variance with the conclusion that those tars having a high sulfonation index hardened more than those with a low sulfonation index. It would seem that the harder residual binders should produce specimens with high stability values, but it is indicated from these stability tests that the inherent bonding strength of the residues from tars with a large amount of unsulfonated residue in the 300° to 355° C. distillation fraction is less than that of tars which have low sulfonation indexes for this fraction.

CONCLUSIONS

This investigation is primarily a study of the sulfonation index test and the effect of the unsulfonated material present in tars on its various properties as shown by laboratory tests. No correlation of this work with the behavior of the tars during construction or under actual service conditions has been made. Consequently, no conclusions can be drawn as to the maximum sulfonation index that is definitely detrimental to the quality of the tar as a road-binding material. It is believed, however, that this investigation justifies the following conclusions:

1. The method of determining the sulfonation index should be modified by decreasing the weight of the sample from 10 to 5 grams and the amount of 37N acid from 30 to 20 milliliters. These modifications increase the proportion of acid to distillate and make possible more vigorous shaking of the sample, which insures more complete sulfonation and more reproducible results.

2. The selective solubility of the distillation fractions in either dimethyl or diethyl sulfate cannot be used as a basis for a quantitative test of the unsulfonated residue to replace the sulfonation index test.

3. There is no direct relationship between the specific gravity of the distillation fraction and the sulfonation index of the tar for that fraction. However, for the tars studied in this investigation and for other tars tested in this laboratory, there are critical values for the specific gravity at 38°/38° C. of the distillate above which the sulfonation index for the particular fraction is less than the maximum requirement usually specified.

4. The critical values for the 0° to 300° C. fraction seem to be about 0.955 at 38°/38° C. for a maximum sulfonation index of 7 and 0.960 at 38°/38° C. for a maximum sulfonation index of 6. For the 300° to 355° C. fraction, the critical values for the specific

gravities at 38°/38° C. were found to be 1.04 for a sulfonation index of 1.5 and 1.05 for a sulfonation index of 1.0.

5. The viscosity coefficient can be used as a basis for comparing the consistency of various materials at a given temperature. The difference in the viscosity coefficient for two residues from the same tar is a measure of the amount of hardening that has occurred.

6. The viscosity coefficient of the residue increases with the amount of loss of distillate, and the average amount of increase for 1 percent of loss is termed "the hardening index." The hardening indexes for the residues from shorter periods of exposure are equal to those for the distillation test but for longer exposures the hardening indexes increase.

7. The difference between the viscosity coefficient of the exposure residue and the viscosity coefficient determined from distillation data for an equal loss is a measure of the internal hardening of the sample. For the samples tested in this laboratory this is in order of the sulfonation indexes, the sample with the highest indexes having undergone the largest amount of internal hardening.

8. The stability developed by weathered sand-tar mixes is generally in inverse order to the amount of sulfonation residue in the 300° to 355° C. fraction of tar binder.

9. The presence of unsulfonated residue in the tar in considerable percentages has an adverse effect on the binding and weathering properties of the tar.

10. Since the present road tar specifications allow wide variations in the percentage of distillate off at the various cutting temperatures and in the softening point of the distillation residue, nonuniform curing and weathering can be expected from road tars meeting the same specification. For this reason the problem of correlating the amount of unsulfonated residue to the behavior of the tars in service is likely to be difficult.

(Continued from p. 156)

that each series represents country-wide totals, that railway travel of the commutation type (daily home-to-business trips) is excluded, the seasonal changes are very similar except for the "long trip" railway travelers home for the Christmas holidays in December and January, which has no counterpart in highway travel. Of course the highway traffic includes trucks but, as indicated in table 1, approximately 78 percent of it is composed of passenger vehicles.

Early in 1942 it was decided to condense the size of reports of traffic recorder operation issued by the Public Roads Administration in order to produce the information more promptly—in the period of greatest value. As a part of the process of condensation and early release of the material it was decided to present a summary which would include all data received from the States within 3 weeks after the end of the period during which the machines were operated. Thus a summary of traffic data for March would be available on April 21, and so on each month. These preliminary figures would be subject to correction a month later when traffic information from nearly all States had been received; and a complete summary for March would be available on May 21. In table 4 is shown a comparison of the partial and complete data for March, April, and May, 1942. Although data from less than half of the States, and a little more than one-third of the traffic recorders, were available within 3 weeks, the partial or preliminary figures for percentages of change have differed by not more than 1 percent from the complete data received a month later.

TABLE 4.—*Comparison of percentage changes in traffic volume as indicated in preliminary and final tabulations, 1942*

Month	Summary of preliminary data			Summary of complete data		
	Number of States	Number of stations	Traffic change	Number of States	Number of stations	Traffic change
			Percent			*Percent*
March	18	179	−11.5	40	452	−11.3
April	20	183	−14.8	42	506	−15.8
May	20	200	−22.6	43	514	−21.7

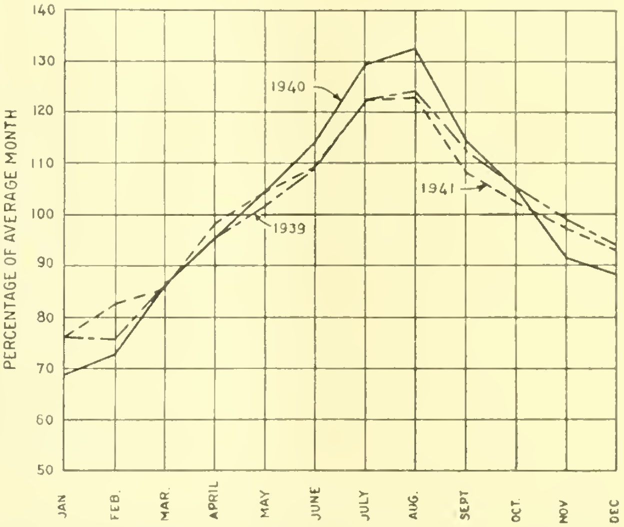


FIGURE 3.—SEASONAL VARIATION IN TRAFFIC RECORDED AT AUTOMATIC TRAFFIC COUNTERS, 1939, 1940, AND 1941.

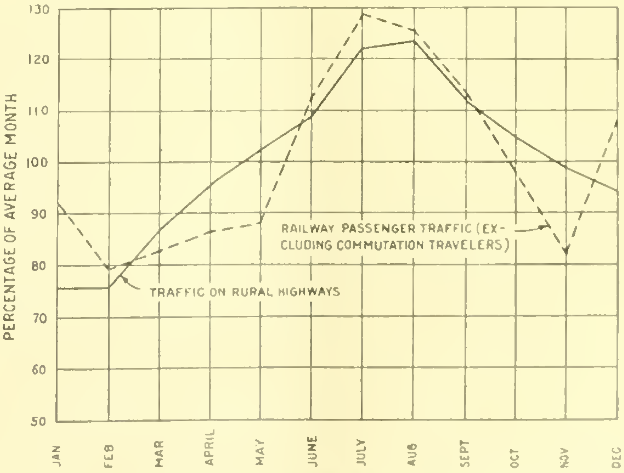


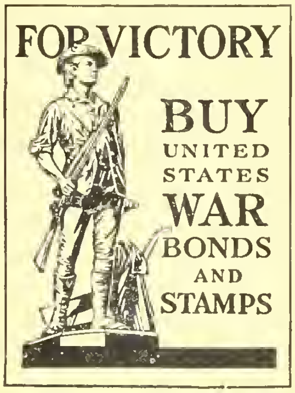
FIGURE 4.—SEASONAL VARIATION IN TRAFFIC ON HIGHWAYS AND RAILWAYS, 1939.

RECENTLY PUBLISHED BULLETINS AVAILABLE

Two bulletins of interest to highway officials have recently been published.

"Moments in I-Beam Bridges" has been issued as Bulletin Series No. 336 of the University of Illinois Engineering Experiment Station. The data reported are based entirely on analytical considerations and give a better understanding of the behavior of the I-beam bridge consisting of a concrete slab continuous over steel beams, a type of structure that has found widespread use for highway bridges. The Public Roads Administration has a limited number of copies of Bulletin No. 336 available for free distribution. Requests should be addressed to the Public Roads Administration, Federal Works Agency, Washington, D. C.

"Stresses in the Corner Region of Concrete Pavements" has been issued as Bulletin No. 157 of the Iowa Engineering Experiment Station. The bulletin reports the results of laboratory stress measurements and field observations, and presents a relatively simple working formula for calculating corner load stresses. Single copies of this 96-page bulletin may be obtained without charge from the Iowa Engineering Experiment Station, Iowa State College, Ames, Iowa.



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PUBLIC ROADS ADMINISTRATION

D. M. BEACH, *Editor*

Volume 23, No. 8 April–May–June 1943

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Commissioner of Public Roads, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

THE STRUCTURAL DESIGN OF CONCRETE PAVEMENTS

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by **L. W. TELLER**, Principal Engineer of Tests
and
EARL C. SUTHERLAND, Senior Highway Engineer

PART 5.—AN EXPERIMENTAL STUDY OF THE WESTERGAARD ANALYSIS OF STRESS CONDITIONS IN CONCRETE PAVEMENT SLABS OF UNIFORM THICKNESS

THIS IS the last of a series of reports of an extensive investigation, undertaken by the Public Roads Administration in 1930 with the general objective of developing information that would be of assistance in better understanding the structural action of concrete pavement slabs. Much of the work has been described in four previous reports published in this same journal, as follows:

Part 1.—A Description of the Investigation, vol. 16, No. 8, October 1935.
Part 2.—Observed Effects of Variations in Temperature and Moisture on the Size, Shape and Stress Resistance of Concrete Pavement Slabs, vol. 16, No. 9, November 1935.
Part 3.—A Study of Concrete Pavement Cross Sections, vol. 16, No. 10, December 1935.
Part 4.—A Study of the Structural Action of Several Types of Transverse and Longitudinal Joint Designs, vol. 17, Nos. 7 and 8, September and October 1936.

Since concrete is a material with recognized elastic properties, engineers concerned with the design of concrete pavement have long searched for a theory that would adequately express the relations between applied forces and the resultant stresses in pavement slabs of this material.

A reliable general theory of slab stresses would serve at least three important purposes. In the first place it would enable the designer to determine the thickness and the form that the slab should have to function without failure under specified conditions of loading and support; second, it would make possible accurate estimates of the loads which might be imposed with safety on existing pavements; and third, it would provide a useful tool for judging the relative effects of vehicle loads of various magnitudes in studies of the costs of providing facilities for vehicles of different types and weights.

From time to time, over a period of many years, theoretical treatments of the load-stress relation in concrete pavements have been offered (*9, 4, 10, 1*).[1] None of the early analyses was general in scope. Rather, each was concerned with some special situation that was assumed to be critical and for which rather broad assumptions were sometimes proposed. The principal weakness consistently seemed to lie in the assumptions that were made regarding the conditions of support, although other assumptions open to serious question were sometimes present.

ELASTIC ACTION OF SUBGRADE AN IMPORTANT FACTOR IN WESTERGAARD ANALYSIS

It was not until the original Westergaard analysis was published that a rational theory of general application became available (*23*). In this treatment, it is assumed that the slab acts as a homogeneous, isotropic, elastic solid in equilibrium and that the reactions of the soil are vertical only and are proportional to the deflections of the slab. The relations between applied loads and critical stresses are then developed on the basis of elastic theory for the three cases of a wheel load applied on the surface of the slab at a free corner, at an interior point and at a free edge (at some distance from a corner) respectively.

In the first paper of this series (*18*) mention was made of the Westergaard analysis and it was stated that one of the objects of the investigation being described was to study the elements and relationships of the analysis by means of load tests on full-size pavement slabs of constant thickness together with such collateral tests as might be found necessary. The investigation was planned in 1929, the sections constructed in 1930 and a considerable amount of testing under load was done during 1931 and 1932. These early tests were referred to by Westergaard in his supplementary paper (*25*) in which he says "The tests suggest certain adaptations of the theory. There will be needed some restatements of analytical results and some supplementing and modification of the theory".

The extension of the original analysis that is contained in this second paper permitted a more comprehensive study to be made and, as a result, much additional information has been developed. The present paper contains a description of the work that was done on this part of the general project, with a presentation of the data obtained and a discussion of their significance.

If one studies the Westergaard analysis he is at once impressed by the importance of the assumption which relates to subgrade support. In the original theory (*23*) it was assumed that the reactions of the subgrade are vertical only and are proportional to the deflections of the slab, the reaction per unit of area at a given point being the product of the deflection at that point and a coefficient of subgrade stiffness, k, which was termed the modulus of subgrade reaction. This modulus is normally expressed in pounds per square inch per inch of deflection (or pounds per inch cube).

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[1] Numbers in italics refer to bibliography at end of article. The above references are in chronological order.

In the supplementary theory (*25*) Westergaard proposed a new coefficient, K, defined by the relation

$$K=kl$$

in which l is a linear dimension, termed the radius of relative stiffness, which appeared in the original theory.

The reason for the proposed new coefficient was an expectation that K would be less dependent on the stiffness of the pavement slab than k. Westergaard stated that "the truth may lie between the two extreme cases of a constant k and a constant K."

To make practical use of the analysis one must be able to assign a value to the modulus of subgrade reaction for the particular soil structure with which he is concerned. At the time the investigation was undertaken no determinations of the value of such a soil coefficient had been made, so there was no background of experience in testing that would indicate either the probable range in values of the coefficient or a procedure by which values might be obtained. Therefore, it was necessary to devise a test procedure that would indicate how the soil of the subgrade beneath the test sections behaved when subjected to pressure intensities and vertical deformations of the same order as occur under pavement slabs in service.

Because of its importance as a part of the study of the Westergaard analysis and because of its current general interest, this study of the elastic action of the subgrade is reported in some detail in this paper, being presented as a separate section, preceding the discussion of the results of the studies of the other relationships expressed in the analysis.

DETERMINATION OF THE MODULUS OF SUBGRADE REACTION

The ideal subgrade assumed by Westergaard is perfectly elastic, has uniform elastic properties at all points and its vertical deformation varies as a linear function of the vertical pressure exerted on its surface. Such a subgrade probably does not exist and the problem becomes one of determining by some test procedure, how nearly the soil under a given pavement approaches the ideal and what stiffness coefficient, if any, can reasonably be assigned to it for the purpose of applying the method of analysis to a particular problem.

While it is a fact that stress values as computed with the Westergaard equations are not particularly sensitive to variations in the value of the subgrade modulus k, if comparisons are to be made between computed stresses and those determined experimentally, the value of the coefficient k used in the theoretical computations must be determined with at least a fair degree of accuracy before dependable values for the computed stresses can be had.

The elastic deflections of a concrete pavement slab of usual design under the action of normal highway loadings are quite small, probably of the order of 0.05 inch or less, depending upon the position of the load, the details of the slab design and other factors that influence deflection. The area over which deflections occur is, on the other hand, relatively large, as may be seen by referring to load-deflection data presented in Part 3 of this series (*20*). In developing a test to be applied to the soil in place for determining the modulus of subgrade reaction, it seemed only reasonable to study the behavior of the soil when subjected to deformations of the same general order of magnitude as would obtain under a pavement slab deflected by a motor vehicle wheel load and, at the same time, to deform the soil over a relatively large area.

THREE METHODS OF MEASURING LOAD SUSTAINING ABILITY OF SOIL DISCUSSED

There appear to be at least three methods or procedures by which the load sustaining ability of the soil can be measured under field conditions. These may be described briefly, as follows:

1. Load-displacement tests in which loads are applied at the center of rigid circular plates of relatively small size, the pressure intensity on the soil being uniform over the entire area of the plate. In these tests the applied load, the mean vertical plate displacement and usually the time intervals are measured.
2. Load-displacement or load-deflection tests in which the load is applied at the center of slightly flexible rectangular or circular plates of relatively large dimensions. In this case some bending of the plate (or slab) occurs and the pressure intensity under the plate is not uniform throughout the area of its contact with the soil. The load, the vertical displacement of various points throughout the area of the plate and possibly time intervals are measured.
3. Load-deflection tests on full-size pavement slabs in which the load-deflection data are obtained by measurement and used in the Westergaard deflection formulas to provide a value for the soil stiffness coefficient or "modulus of subgrade reaction."

If all three methods were equally satisfactory the first procedure offers the practical advantage of requiring test loads of lesser magnitude with a corresponding reduction in the size of the equipment. Also the number of measurements is less since no plate deflections are involved. Its use is complicated, however, by two conditions. The first is that the ability of a soil to sustain a given unit pressure varies within limits with the area over which the pressure is applied to the soil. This variation may be quite marked and this makes it necessary to determine the effect of size of plate in order to avoid error from using a bearing plate that is too small. The second complication is that the supporting ability of a soil varies with its moisture state and it is necessary, therefore, to take special precautions to insure that the soil on which the bearing plate is placed is in the same physical state and moisture condition as that which will obtain or does obtain under the pavement to be considered.

The second procedure has a certain theoretical appeal but offers considerable practical difficulty as a method of test. In this procedure the plate is deflected by the centrally applied load much as the pavement slab deflects under the action of a wheel load. The shape of the deflected plate must be determined precisely and its vertical displacement measured in order to be able to estimate accurately the volumetric displacement of the soil that is effected by the application of the test load on the plate. The modulus of subgrade reaction is then computed by dividing the load (in pounds) by the volume of displaced soil (in cubic inches). Some use has been made of this procedure in England (*12*). The possibilities of the method should be more thoroughly explored.

When using the third method, the procedure is to apply test loads at the free corner, free edge or interior point of a pavement slab of uniform thickness and of normal size. If the elastic modulus of the concrete in the slab is known, for the moisture and other conditions that obtain, it is possible to determine the value of the effective modulus of subgrade reaction from the maximum slab deflection under the applied load by means of the deflection formulas given by Westergaard in his supplementary paper (*25*). This method for determining the soil stiffness coefficient will be discussed further in a later part of this report in connection with the presentation of deflection data obtained in such tests. The remaining part of this section of the report will be devoted primarily to a discussion of work done with the first method, i. e., load-displacement tests with rigid plates of relatively small size.

Load-displacement tests with rigid plates have been made many times in the past in studies of the bearing capacity of soil for foundations. The data obtained in such tests are not applicable to the problem of pavement support, however, because of the conditions surrounding the tests and the extent to which the soil deformations were carried.

LOAD-DISPLACEMENT TESTS WITH RIGID PLATES

When a rigid plate, supported by soil, is subjected to a vertical force or load applied at its center, the soil deforms and the plate moves downward. This downward movement of the plate under load has been variously termed deflection, displacement, penetration, settlement or subsidence, and tests which make use of this action have been termed accordingly load-deflection—or load-subsidence tests. In this discussion the terms displacement and load-displacement tests will be used.

The load-displacement tests which were a part of this general investigation comprise four series, as follows:

Series 1.—Five circular bearing plates and one square bearing plate were used. The diameters of the circular plates were 8, 12, 16, 20, and 36 inches respectively and the square plate had 48-inch sides. The magnitudes of the loads applied to each plate were such as would cause displacements within the range 0.01 to 0.05 inch, this being the approximate range of concrete pavement slab deflections under the action of legal maximum wheel loads.

The tests of this series were divided into two parts. In the first part, all plates were placed successively, in the descending order of size, at the same location on the subgrade while in the second part of the series the test with each plate was made at a different location but within the same general area.

Series 2.—Eleven circular plates, having diameters of 2, 4, 6, 8, 12, 16, 20, 26, 36, 54 and 84 inches respectively, were used. As in series 1, the applied loads were such as to cause displacements within the range 0.01 to 0.05 inch. In series 2 the tests with each size of plate were made at a separate location on the subgrade.

Series 3.—These tests were the same as those of series 2 except that the maximum plate displacement was increased to approximately 0.25 inch, and a somewhat different loading procedure was followed.

Series 4.—Only the 54-inch diameter plate was used. The plate remained at one location and the displacements were kept within the range 0.01 to 0.05 inch as in series 2. The loads were applied, however, in June and in January.

The first tests made in this study were those of series 1. They were intended to explore the effect of size of bearing plate on the load-displacement relation and also to compare data obtained in a series of tests at one location with those from tests that were identical except that each test of the series was made at a different location in the same general area. Obviously it would be preferable to have each test made in an area undisturbed by previous loading, provided there was sufficient general uniformity in the soil structure to eliminate the possibility of local variations in structure affecting certain tests (particularly those in which small bearing plates were used).

The tests of series 2 were designed to extend considerably the data from series 1 on the effect of size of the load-displacement relation.

The tests of series 3 were to provide information on the comparative supporting ability of the soil when subjected to deformations greater than those which usually occur under rigid pavements. These data extended the range of the information obtained and permitted certain comparisons with other data to be made.

Series 4 was simply a study of the effect of the annual change in the physical state of the soil directly under the slab on its ability to support load. For this reason only the 54-inch diameter bearing plate was used and the displacements were limited to the 0.01 to .005 inch range.

METHODS OF TESTING DESCRIBED

All of the soil tests of the four series were made on a part of the originally prepared subgrade that was reserved for this purpose. The soil was described as a uniform brown silt loam (classification A-4) and detailed information concerning it is contained in the first report of this series (*18*).

When making tests such as those described it is important that the soil on which the bearing plates are placed be in the same physical state as that under the pavement to which the tests are to be related. An effort was made to accomplish this by casting the larger bearing plates (which were of concrete) on the subgrade several months in advance of the first scheduled tests and by casting, at the same time, a number of concrete slabs, each 4 feet square, at those locations where tests with the smaller bearing plates were to be made later. By this means the soil was given the same protection and the same opportunity for moisture equilibrium was afforded as obtained with the test pavement itself.

The larger plates of concrete were left in place and loads were applied at the scheduled time. The smaller plates were of steel and with these the procedure was different. At the proper time the small concrete protecting slab was removed and waterproof paper spread over the area where it had been. A thin layer of portland cement mortar or of plaster of Paris was then spread over the waterproof membrane and the bearing plate was bedded in this mortar. This procedure gave a uniform contact between the soil and the bearing plate yet prevented moisture from the mortar from entering the soil. Figure 1 shows one of these smaller

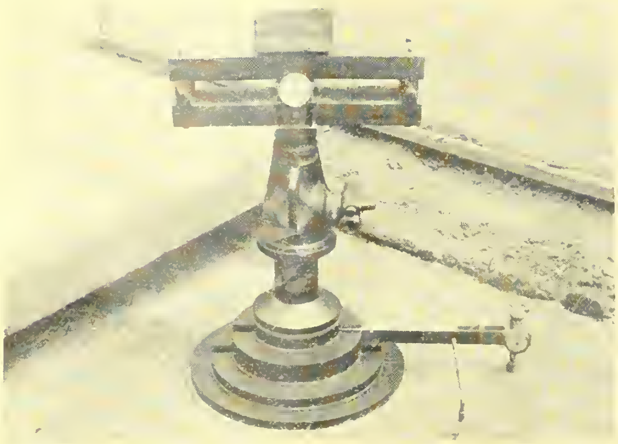


Figure 1.—Determining the Modulus of Subgrade Reaction With a Small Bearing Plate. The Small Concrete Slab That Covered the Subgrade Has Been Turned Back. Displacement Is Measured With a Clinometer.

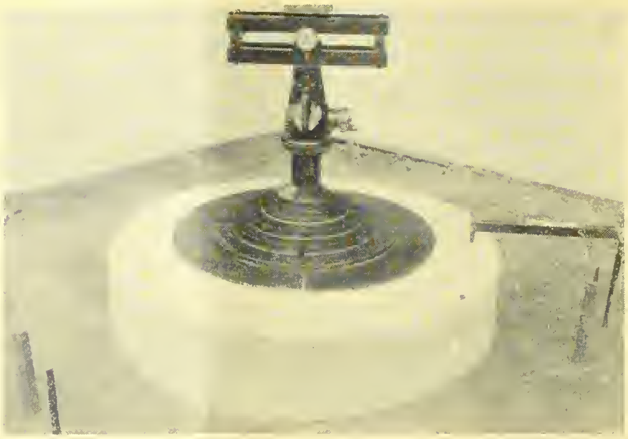


Figure 2.—Determining the Modulus of Subgrade Reaction With a Large Bearing Plate. The Bearing Plate Was Cast on the Subgrade Some Time Before Testing. Displacement Is Measured With a Clinometer.

plates in place while figure 2 shows a test with one of the larger plates.

The vertical loading force was applied by a jack reacting against a large horizontal cylindrical tank mounted on a dolly frame and filled with water. The magnitude of the force was determined with a dynamometer. All of this equipment is described in the first report (*18*). The vertical movements of the bearing plate were measured either with dial micrometers supported on a bridge or by a series of clinometer measurements in the manner described in the first report (*18*). When using the clinometer it was necessary to place the reference point at some distance from the bearing plate and to carry the level line over a series of intermediate points to the bearing plate so that soil movements in the vicinity of the bearing plate would not cause errors in the displacement data. With the smaller bearing plates the mean displacement was obtained from a single point in the center of the plate while for the large plates the displacements of three points symmetrically spaced along the perimeter were measured and averaged. All of the plates were sufficiently rigid to prevent appreciable bending as used.

In the tests of series 1, 2 and 4 an effort was made to deform the soil in a manner simliar to that which might be expected under a concrete pavement. A comparatively large number of loadings were applied in each test and the magnitudes of the plate displacements were kept within the normal load-deflection limits of a concrete pavement slab.

For each size of bearing plate a series of 3 to 5 ascending load values was selected, such that the series would give a good spread of displacement values and the maximum would not produce a displacement greater than the desired limit. With the smallest load value selected for a plate of a given size, the load was applied and removed several times. The number of applications was not constant but was determined by the character of the data, it being desired to reach a condition such that each succeeding application of a given load would produce the same vertical displacement of the bearing plate. This might be termed a state of approximate elastic equilibrium. The number of loadings required to develop this condition, with the soil on which tests were made, usually varied from about 5 to 10. When a satisfactory load-displacement relation had been determined for the lowest load value, the procedure was repeated with the next higher load value and so on until the displacement limit was reached. As stated previously, when the tests with plates of various diameters were to be made at one location, the procedure was to test with the largest plate first then with the next largest and so on.

When a load is applied to a bearing plate a displacement of the bearing plate begins and, under some conditions, may continue for a long time before a state of complete equilibrium is reached. Similarly when the load is removed a certain amount of elastic recovery occurs and this too may continue for some time. As a practical matter it is not possible to continue each test cycle until the last vestiges of either displacement or recovery have disappeared before proceeding with the next loading cycle. After some experimentation it was decided that for the soil condition, load intensities and sizes of bearing plates in this investigation, a condition of essentially complete equilibrium would be reached if each load was maintained for 5 minutes after reaching its full magnitude and if after the complete removal of that load 5 minutes elapsed before the application of the next load.

The procedure followed in the tests of series 3 was somewhat different, as mentioned previously. As the data from this series were, in part, to be compared with data developed in tests of series 2, the tests of series 3 were made immediately after the completion of those of series 2 so that no change in the condition of the subgrade soil could occur between the two series. In order to obtain load-deformation data for the soil in question which might be compared directly with those obtained by other agencies for other soils, the soil deformation limit was increased to approximately 0.25 inch for this series and the loading procedure was modified to conform more closely to that followed in the tests made by others. In the tests of series 3 only one load of each magnitude was applied. This load was left on for 5 minutes, removed completely and a period of 5 minutes allowed to elapse before the next larger load was applied. The maximum displacement limit of approximately 0.25 inch applied only to the smaller sizes of bearing plates since the maximum reaction possible with the loading equipment used was only about 50,000 pounds and this was insufficient to cause a dis-

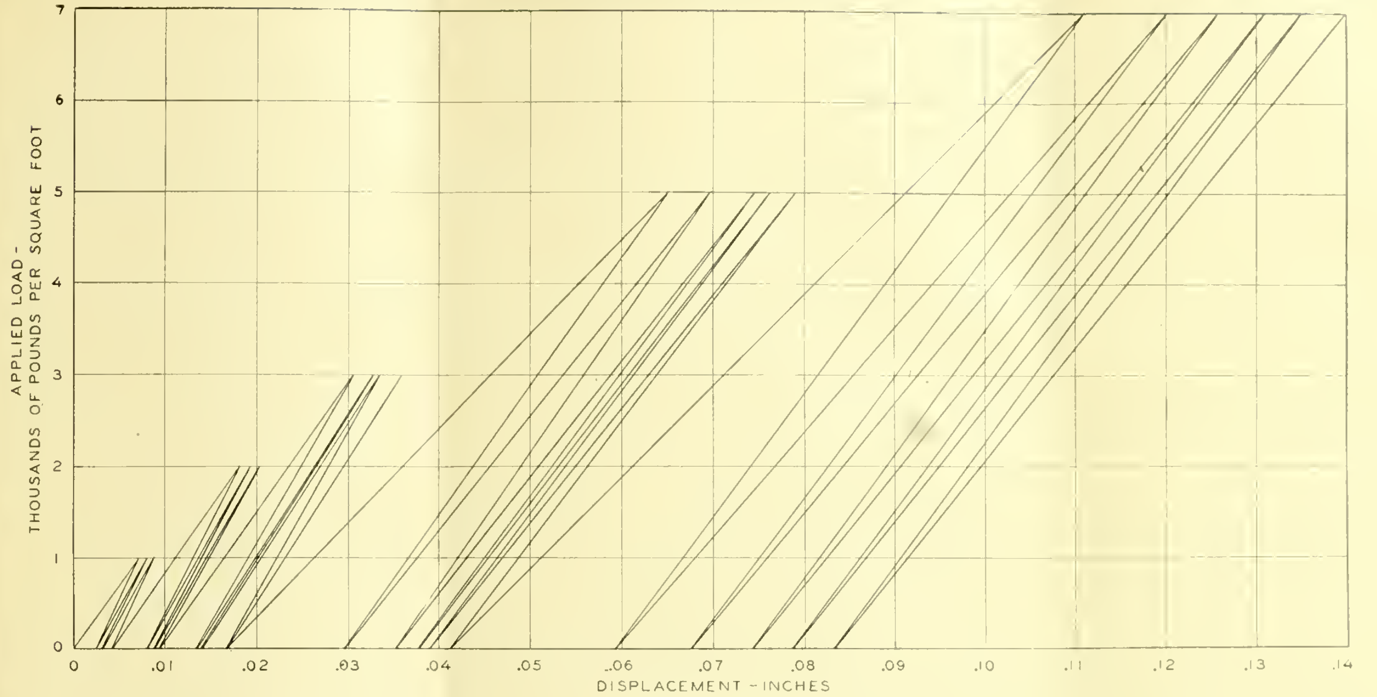


Figure 3.—Typical Load-Displacement Data for a 26-Inch Diameter Bearing Plate Plotted With Respect to the Initial Position.

placement of the desired magnitude with the larger plates.

Information regarding the moisture condition of the soil was obtained from samples taken from under a 4- by 4-foot concrete slab immediately adjacent to the point at which the bearing test was being made. The soil samples were taken just before and just after the bearing test. The moisture content was determined by weighing, drying, and reweighing.

SOILS TESTED SHOW HIGH PERCENTAGE OF RECOVERY AFTER REMOVAL OF TEST LOAD

When a bearing plate is subjected to a sequence of loads in the manner described for series 1, 2, and 4, data of the type shown in figure 3 result. This graph shows a log of the progressive displacements of a 26-inch diameter plate caused by a series of loadings of five magnitudes, the load of each magnitude being repeated several times, as described earlier in the report. The mean plate displacements are shown throughout both with respect to the original plate position and to the position just before the particular load was applied. The graph shows also the magnitude of the recovery during the 5 minute period following the removal of each load.

The data obtained in these load-displacement tests showed that, for a given plate size and load intensity, the magnitude of the displacement usually decreased somewhat with each load application until several loads have been applied, after which the load-displacement relation remained fairly stable. As soon as the pressure intensity was increased by the application of a greater force to the plate, the same repetition of loadings was necessary to again bring about the condition of approximate elastic equilibrium. Data of this type and extent were obtained for each of the various sizes of bearing plate listed earlier.

It is apparent in the graph that for the conditions of soil, pressure intensity, soil deformation, and time which obtained in this test, the action was never completely elastic. This is evidenced by the residual deformation after each loading. A study was made of the data to determine the percentage of recovery that followed the complete removal of the load and the result of this study is summarized in table 1.

Table 1.—*Average recovery of soil after removal of test load*

Displacement	Diameter of bearing plate in inches—			
	4	20	54	84
Inches	*Percent*	*Percent*	*Percent*	*Percent*
0.010	93	90	93	89
.020	90	90	95	97
.050	90	91	95	99

The percentages of deformation recovered upon removal of the load, as shown in this table, are values determined by averaging the movements after preliminary loadings had developed a state of approximate elastic stability.

The data indicate that the percentage of recovery was rather high in all cases, that it was greater with large plates than with small ones and that it tended to be greater for the larger plate displacements than for the smaller ones.

EFFECT OF SIZE OF BEARING AREA STUDIED

Since the recovery is not complete, there is a cumulative residual or permanent displacement as the test proceeds. For the soil on which these tests were made, the magnitude of the cumulative deformation appears to vary with the number of load applications and with the pressure intensity used.

This gradual settlement of the bearing plates under the succession of test loads raised the interesting question as to what effect such a yielding of the soil might have on the load-strain relation of a concrete pavement slab. To obtain information on this point, tests were made on a full-size pavement slab on the same subgrade that was used for the soil bearing tests. In the tests on the pavement slab a load was applied a relatively large number of times at the corner, the edge and the interior and the critical strain measured for each load application. It was found that at the corner there was a progressive increase in the strain caused by a given load, but at a diminishing rate, until there had been about 70 repetitions of the load. After this there was apparently no increase in the critical strain. The maximum increase in strain from the initial to the final loading was approximately 10 percent. Repetition of load at the free edge and at an interior point caused no appreciable change in the critical strains. The data from these load strain tests are presented in graph form in a later section of this report.

It will be recalled that in series 1 tests were made with various sizes of bearing plates, both at a common location and at individual locations. The tests with each size of plate provided data of the type shown in figure 3. The data from all of the individual tests of series 1 are summarized in figure 4 which shows load-displacement relations for each of the several sizes of bearing plate, both for the tests at a common location and for those at individual locations. The displacement values in this figure are with respect to the position of the plate immediately before the application of the particular load, being in each case the average of 2 or 3 determinations made after the subgrade had attained a condition of approximate elastic equilibrium for the particular displacement. No tests were made at the common location with the 36-inch round plate and the 48-inch square plate.

The data obtained in this series of tests afforded two significant comparisons. They showed first the important effect of plate area on the pressure intensity required to produce a given plate displacement on the soil in question. A similar effect has been observed by other investigators in tests where much larger plate displacements were used. This effect will be discussed in more detail later in this report. The second comparison was between the tests with a series of plate sizes at one location and a similar series at individual locations. For this comparison the data show that for a given plate size (within the 8- to 20-inch diameter range) essentially the same load intensity-plate displacement relation obtained. It was considered better procedure, however, to make the tests with each plate size at individual locations in the subsequent program of series 2.

In figure 5a and 5b the same data are arranged in a different manner. In figure 5a are shown the relations between pressure intensity and the diameter of the bearing plate for three magnitudes of plate displacement, 0.01, 0.02, and 0.05 inch, respectively. This graph brings out forcibly the importance of both the area and the degree of the displacement in determining the load supporting ability of the soil. The shape of the curves, shown in this graph, is generally similar to that found in other tests, both in this country and in Europe, in which loads were applied to rigid plates of various sizes that were supported by soils not strictly granular in character (*3*, *5*, *6*, *8*, *2*). Generally speaking, the other tests cited differed from those described in this report in two important particulars. In the first place, the soil deformations were carried considerably farther than in the present study and, in the second place, the maximum dimension of the bearing plate, i. e., the diameter of the circular plate or the side of a square plate was, with one exception, less than 42 inches.

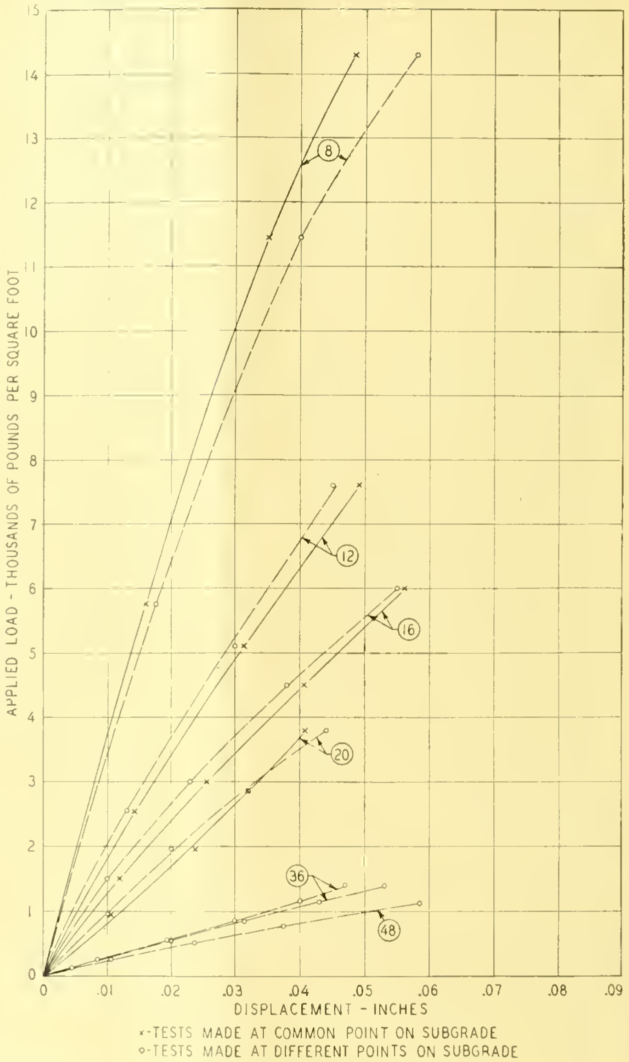


Figure 4.—Load-Displacement Relations for the Several Bearing Areas of the First Series of Tests. Figures in Circles Indicate the Diameter of the Bearing Plate in Inches.

In figure 5b the data from series 1 are again shown but, in this case, with the pressure intensity related to the perimeter-area ratio, an inverse function of the plate size, in the manner suggested by Housel (*6*). Three curves, one for each of the three magnitudes of plate displacement used in the tests, are given in the graph.

In both figures 5a and 5b the data obtained with the 48-inch square plate are included, being plotted on the basis of area, the shape factor being ignored. Thus there may be reason to question the accuracy of the points which are plotted on the 54-inch diameter and 0.083-inch^{-1} perimeter-area ratio ordinates respectively in the two parts of this particular figure. Data obtained in the subsequent series of tests support the relations shown in figures 5a and 5b, however.

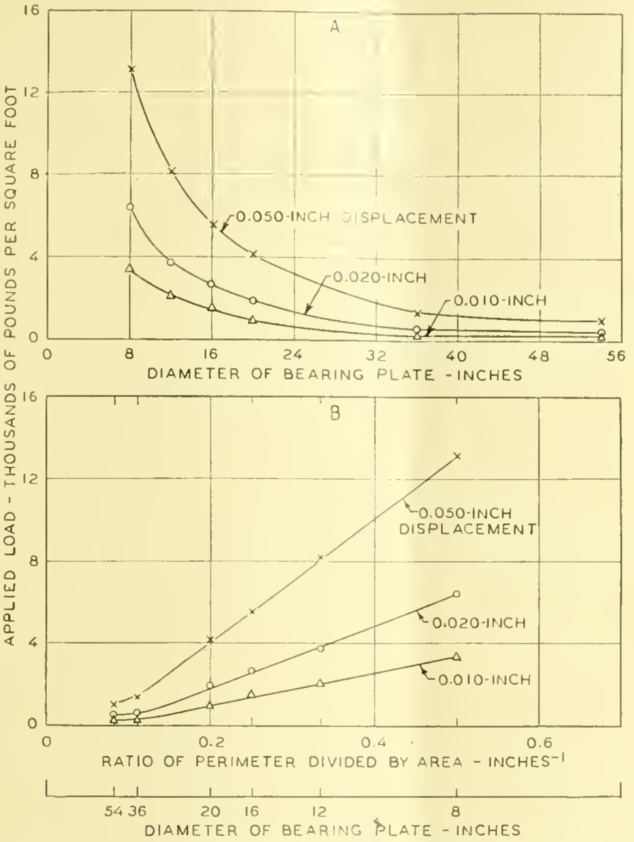


FIGURE 5.—EFFECT OF BEARING PLATE SIZE ON THE LOAD REQUIRED TO PRODUCE A GIVEN PLATE DISPLACEMENT. DATA FROM SERIES 1.

It is of interest to note that, when the data are plotted in the manner shown in figure 5b, an essentially straight line relation exists between the pressure intensity and the ratio of perimeter to area for plates of 20-inch diameter and less while for the larger plates used in this series, it appears that the linear relation does not hold. The use of the inverse function of plate size $\left(\frac{2}{\text{radius}}\right)$ for the abscissas results in such a convergence of this scale near the origin that it is difficult to determine at what point on these curves the departure from linearity begins but it appears to be in the vicinity of 30 inches on the diameter of bearing plate scale.

SMALL PLATES FOUND UNSUITABLE FOR USE IN DETERMINING RELATION BETWEEN PRESSURE INTENSITY AND PLATE DISPLACEMENT

It will be recalled that, in series 2 and series 3, tests were made with eleven sizes of circular bearing plates ranging from 2 to 84 inches in diameter. In series 2 the loading procedure and displacement ranges were the same as in series 1, while in series 3 the loading procedure was different and the displacement range was increased consistently.

In figures 6 and 7 are summarized the load displacement data obtained in the tests of series 2 and series 3 respectively. These graphs correspond to figure 4 which contains similar data from the tests of series 1. Attention is called to the difference in the horizontal scales used in the two figures, this being necessary because of the difference in the displacement range of the two series of tests. It will be noted that in series 3 the displacements are limited to values less than the maximum in the case of the large bearing plates. With the large areas, the available load reaction of approximately 50,000 pounds limited the pressure intensity (and the plate displacement) which could be obtained.

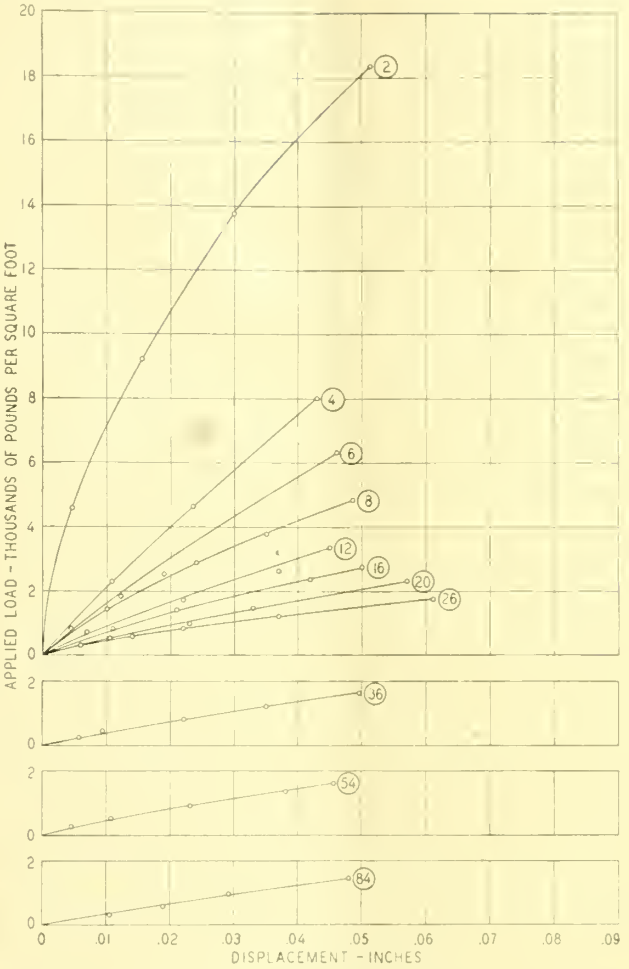


FIGURE 6.—LOAD-DISPLACEMENT RELATIONS FOR THE SEVERAL BEARING AREAS OF THE SECOND SERIES OF TESTS. FIGURES IN CIRCLES INDICATE THE DIAMETER OF THE BEARING PLATE IN INCHES.

Figure 8 shows the load-displacement data from the tests of series 2, plotted in the same manner as was used in figure 5a for series 1 in order to bring out the effect of the size of the bearing plate on the ability of the soil to resist deformation. As in the corresponding earlier graph the relation is shown for each of the three displacement magnitudes, 0.01, 0.02 and 0.05 inch respectively.

Figure 9 is similar to figure 8 but contains data from the tests of series 3. Because of the larger displacements in the tests of series 3, the action of the soil was somewhat different and this difference was most marked when the 4-, 6-, and 8-inch diameter bearing plates were employed. Generally speaking, however, the relations between plate size and the pressure intensity necessary to produce a given displacement were similar to those developed by the tests in which the displacements were limited to the small values of 0.01, 0.02 and 0.05 inch, as shown in figures 5a and 8.

The data consistently show that, for the conditions that existed in the present tests, the effect of plate size on the pressure intensity-plate displacement rela-

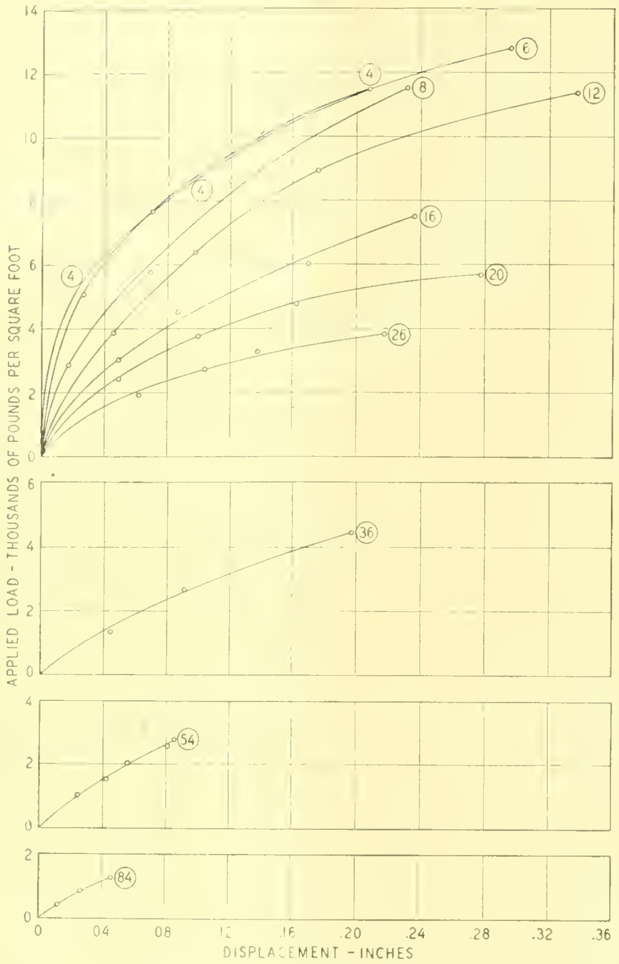


Figure 7.—Load-Displacement Relations for the Several Bearing Areas of the Third Series of Tests. Figures in Circles Indicate the Diameter of the Bearing Plate in Inches.

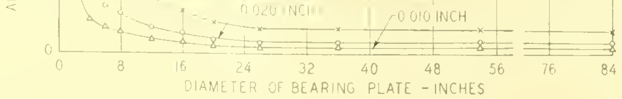


Figure 8.—Effect of Bearing Plate Size on the Load Required to Produce a Given Plate Displacement. Data From Series 2.

tion is not important for plate diameters of about 26 inches or larger, but for plates of less than the 26-inch diameter there is an effect which becomes increasingly large as the plate size diminishes.

Certain earlier investigators, (*3*) and (*5*), concluded that, for soils of the cohesive type, a given pressure intensity applied to bearing plates of various sizes might be expected to produce vertical soil deformations which would be directly proportional to the square root of the areas of the plates. An analysis of the present data indicates that the relation just stated applies quite well for bearing plates having diameters of 26 inches or less, but that for larger plates it does not apply, the magnitude of the divergence increasing rapidly as the size of the plate is increased. For a plate 84 inches in diameter the measured displacement was approximately one half of that which would be predicted if the above-mentioned relation held true.

It should not be assumed that the relations found in the bearing tests at Arlington are applicable generally. Further tests are needed on other soils in place, particularly tests that include large bearing areas, before any attempt can be made to generalize on the relations. It should be remembered also that the studies at Arlington were made primarily to determine the behavior of soil undergoing relatively small deformations and the magnitude of the deformation is shown to be an important variable in the test.

The data obtained are important, however, in showing the size of plates which must be considered in studies of the soil condition of the Arlington pavement research and furthermore because they indicate the necessity for a knowledge of the effect of plate size when making bearing tests with other soil conditions.

SEASONAL VARIATION IN SUBGRADE SUPPORT STUDIED

It will be recalled that the modulus of subgrade reaction is a stiffness coefficient which expresses the resistance of the soil structure to deformation under load in pounds per square inch of pressure per inch of deformation (in the direction of the loading force). Certain pertinent facts have been brought out. It has been shown that, (1) the soil structure is imperfectly elastic; (2) the elastic behavior of the soil is affected by its moisture state; (3) the load resistance of the soil structure, i. e., the pressure intensity required to produce a certain deformation, depends upon the magnitude of the deformation and, within certain limits, upon the area over which the pressure is applied to the soil structure. It is evident that these conditions place limitations on the manner in which the stiffness coefficient can be determined and on the extent to which it can be applied. However, it is believed that, for soils of cohesive character at least, it is possible to obtain approximate but usable values for the coefficient from load-displacement tests with rigid bearing plates on the soil in place, provided certain precautions are taken to minimize the effect of the disturbing influences mentioned above. A study of the data suggests the nature of the precautions to be taken.

If values of the stiffness coefficient k are calculated from load-deformation data, such as those shown in figure 3, it will be found that the value of the coefficient varies with the size of the plate used in the test and with the magnitude of the soil deformation. This has been mentioned previously and is indicated by the relations shown in figures 8 and 9. An analysis of the data obtained in the more comprehensive tests of series 2 was made to determine the extent and characteristics of the variations in the value of the coefficient for the conditions that obtained in this series of tests. Values were calculated for each size of bearing plate used and for soil deformations of 0.01, 0.02, and 0.05 inch for each plate size. The variation in the value of the coefficient with plate size and with the magnitude of the plate displacement or soil deformation is shown in figure 10. From this figure it appears that when making tests to determine the value of the soil stiffness coefficient k

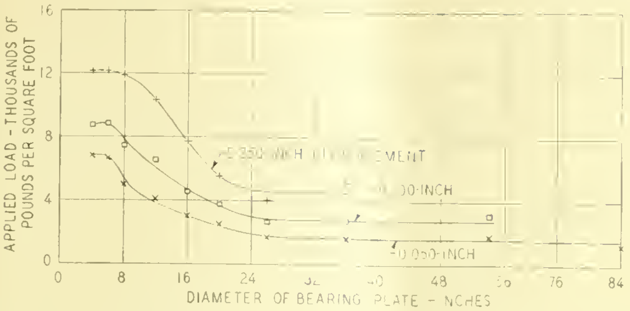


FIGURE 9.—EFFECT OF BEARING PLATE SIZE ON THE LOAD REQUIRED TO PRODUCE A GIVEN PLATE DISPLACEMENT. DATA FROM SERIES 3.

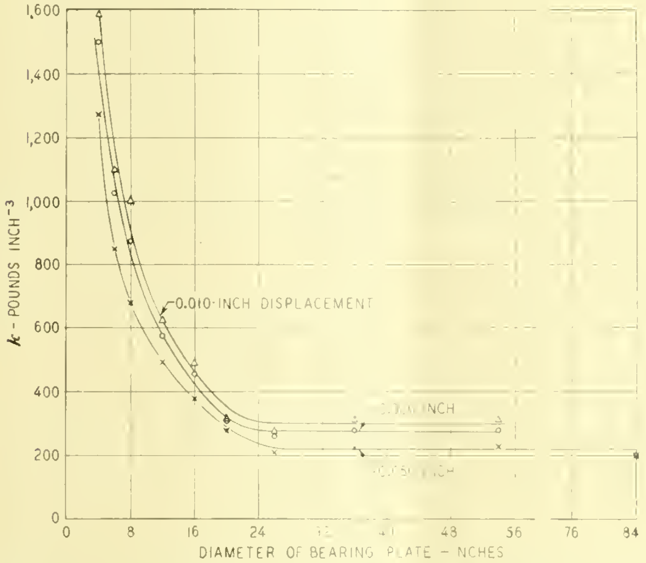


FIGURE 10.—VALUES OF k DETERMINED FROM BEARING TESTS MADE ON CIRCULAR PLATES OF VARIOUS DIAMETERS. DATA FROM SERIES 2.

it is necessary to limit the deformation to a magnitude within the range of pavement deflection and that it is of great importance to use a bearing plate of adequate size. For the conditions of the tests of series 2 the minimum plate diameter is indicated as being about 26½ inches. It should not be concluded, however, that this size of bearing plate would necessarily be adequate for other soil conditions. There is, in fact, evidence that it would not be. This point will be referred to again later.

A similar analysis was made of the data obtained in the tests of series 3. It will be recalled that in this series larger displacements were included and only one load of each magnitude was applied. The values of the coefficient for the various conditions of the tests of series 3 are shown in figure 11.

A comparison of figure 11 with figure 10 shows that the general indications, mentioned above, as to the effect on the value of the stiffness coefficient of the magnitude of soil deformation and of the size of the bearing plate are supported by the data from the tests of series 3 as well as those of series 2. Furthermore, in the one case where the two series are most nearly comparable, i. e., the tests with a limiting soil deformation of 0.05 inch, the agreement between the values obtained in series 2 and series 3 is rather good.

The shapes of the curves shown in figure 11 are different from those in figure 10, particularly in the region of the small bearing plates. This is attributed to the difference in test procedure used in the two series of tests as mentioned in connection with the discussion of figures 8 and 9.

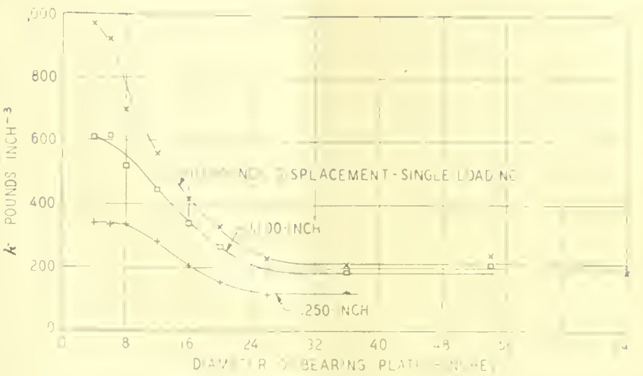


FIGURE 11.—VALUES OF k DETERMINED FROM BEARING TESTS MADE ON CIRCULAR PLATES OF VARIOUS DIAMETERS. DATA FROM SERIES 3.

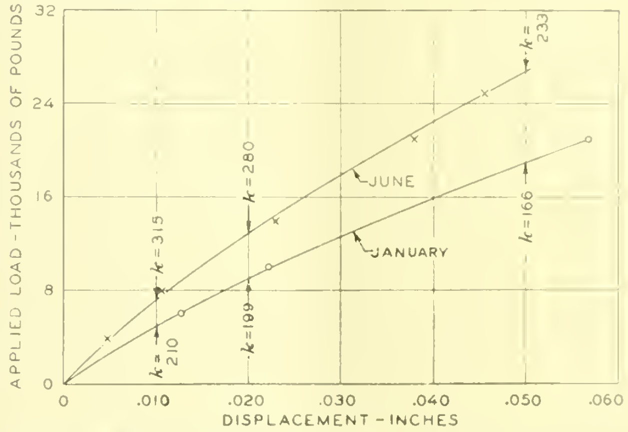


FIGURE 12. EFFECT OF SEASONAL MOISTURE CONDITION AND OF SOIL DEFORMATION ON THE LOAD SUPPORTING ABILITY OF THE SUBGRADE. DATA FROM SERIES 4.

The tests of series 2 and series 3, from which the data shown in figures 10 and 11 were obtained, were made during the summer. The moisture content of the soil was determined to a depth of about 12 inches and was found to be quite uniform, averaging 17 percent.

Because the moisture content of the soil beneath the pavement was known to vary from summer to winter, load-displacement tests with the 54-inch diameter bearing plate were made under both normal summer and normal winter conditions. This made possible some study of the effect of seasonal moisture change on the value of the soil stiffness coefficient.

In these tests, which were designated series 4, the bearing plate remained in position on the subgrade for some time before the test so that moisture conditions were stabilized at the time of test. For the summer condition the moisture content of the soil was uniformly 17 percent to a depth of about 12 inches while for the winter condition it was about 25 percent in the 0- to 6-inch zone and about 19 percent in the 6- to 12-inch zone.

The loading procedure in series 4 was the same as in series 2.

Load-displacement data from the tests of series 4 are shown in figure 12 for both the summer and the winter conditions. On the graph are shown also values of the modulus k calculated from data obtained in the bearing plate tests for soil deformations of 0.01, 0.02 and 0.05 inch, respectively. It is apparent from this figure that the soil structure in question was much more resistant to load deformation under the conditions which prevailed when the summer tests were made than it was at the time of the winter tests, the increase in stiffness from winter to summer being roughly 40 to 50 percent. Undoubtedly the magnitude of the seasonal change in subgrade support will vary with the characteristics of the soil material and with the temperature and moisture changes that occur. Under favorable conditions the variation may be much smaller than that cited. It is obviously important, however, to make sure that the soil at the site of the bearing test is of the same character and density and is in the same moisture state as the soil supporting the pavement under consideration. The important influence which the magnitude of the soil deformation has on the value of the modulus k is further emphasized in this graph.

CAREFUL PROCEDURE NECESSARY IN DETERMINING MODULUS OF SUBGRADE REACTION FROM LOAD-DISPLACEMENT TESTS

The experiments with rigid bearing plates have led to the conclusion that approximate but usable values for the modulus of subgrade reaction as used in the Westergaard equations can be obtained by means of load-displacement tests with rigid bearing plates. The criterion by which the usability of the modulus values determined in this manner is judged has been a comparison of calculated and measured strains in full-size pavement slabs. Since the modulus of subgrade reaction k is but one of several coefficients that must be determined before the comparison can be made the discussion of this comparison will appear in the more general section to follow.

The studies have indicated that there are certain limitations which must be recognized, certain precautions which must be taken and certain procedures which are desirable when determining values for the modulus of subgrade reaction by means of load-displacement tests with rigid bearing plates. These will be mentioned briefly under appropriate headings.

Condition of the Soil at the Site of the Test.—It is obvious that if the data are to reflect the supporting properties of the subgrade, the soil at the site of the test should be truly representative of that subgrade. Not only should the soil materials be the same, but the structure, density and moisture state must likewise be duplicated. It appears that this can be accomplished best by giving the soil to be tested the same compaction as that given the subgrade, by covering the soil to be tested with a concrete slab and by permitting sufficient time to elapse for the development of a stable condition of soil mositure before testing. Unless these precautions are taken, there is no assurance that the soil structure tested was representative of that under the pavement. In the case of an existing pavement these conditions can be satisfied by removing a small section of the pavement and making the bearing plate test on the subgrade itself. Both procedures have been used in the Arlington tests and both are believed to be satisfactory.

Bearing Plate.—Certain physical characteristics of the bearing plate must be considered. It was shown that, within limits, the area of the plate had a very marked effect on the value of the modulus k as determined by the bearing plate test and furthermore that the value which was obtained with data from tests with relatively large plates could not be predicted from similar data obtained in tests with small plates. Also there is evidence that the minimum size of plate that will give satisfactory data depends upon the soil structure being tested. It is important, therefore, to determine by tests with plates of several sizes the minimum size that will be satisfactory for a given soil condition. If this is not possible an expedient would be to test with a relatively large plate, a 48- to 60-inch diameter being suggested by the data at present available.

The rigidity of the plate is apparently an important factor. The use of slightly flexible plates may be feasible but the degree of flexibility is another influence to be considered and sufficient data are not yet available to permit comparisons to be made of the relative value of rigid and slightly flexible bearing plates. Until further studies are made of the use of slightly flexible plates it is believed that rigid plates should be used exclusively, a rigid plate being arbitrarily defined as one which, under the conditions of the load-displacement test, does not deflect or "dish" from perimeter to midpoint by more than 0.004 inch. (NOTE.—The 54-inch diameter bearing plate used in these tests was made of concrete and was 12 inches in thickness. Under a 15,000-pound total applied load, the maximum measured deflection of a diameter of this disc was 0.0034 inch.)

The effect of the shape of the bearing plate is another element which has not been studied adequately. Until more information is available it seems advisable to make all bearing tests with circular plates.

Range of Plate Displacement.—It has been shown that the magnitude of the soil deformation (or plate displacement) had an important effect on the soil stiffness coefficient in the tests at Arlington. How great this effect might be with soil structures radically different from the subgrade under the experimental pavement is not known. It is believed, however, that the maximum displacement of the bearing plate in tests to determine the modulus of subgrade reaction k should not exceed the average deflection of the pavement slab under the expected wheel load. In the absence of measured values, it is probably safe to assume a value within the range 0.02–0.03 inch for this purpose.

Bedding the Bearing Plate.—Because of the small maximum displacement it is especially important that the bearing plate be carefully bedded on the soil under test. The procedure followed in the tests previously described appears to be one way by which a satisfactory initial contact can be obtained.

Loading Procedure.—It seems desirable to have the soil structure in as nearly an elastic condition as possible when the modulus of subgrade reaction is determined and it appears that in the tests at Arlington this end was substantially attained by the procedure adopted. It will be recalled that in this procedure a sequence of loads of equal magnitude was applied to the bearing plate until the soil structure reached a condition such that each successive load produced essentially the same plate displacement and elastic recovery in a given time interval.

Other Tests.—In conjunction with a subsequent program of studies of the structural action of certain types of transverse joints, there was afforded an opportunity to make tests that provide limited comparisons between values of the modulus of subgrade reaction as determined by pavement slab deflections and values obtained by bearing plate tests for a somewhat different soil structure. These comparisons will be discussed later in the more general section of this report.

LOAD-DEFLECTION AND LOAD-STRESS RELATIONS FOR PAVEMENT SLABS OF UNIFORM THICKNESS

WESTERGAARD EQUATIONS FOR DEFLECTION AND STRESS

The Westergaard equations for deflection and for stress in concrete pavement slabs are developed for three cases of load position, as follows:

Case I in which "a wheel load acts close to a rectangular corner of a large panel of the slab" (*23*), referred to as a corner loading.

Case II in which "the wheel load is at a considerable distance from the edges," referred to as an interior loading.

Case III in which "the wheel load is at the edge but at a considerable distance from any corner," referred to as an edge loading.

In the original analysis it was assumed that the slab acts as a homogeneous, isotropic, elastic solid in equilibrium and that the reactions of the subgrade are vertical only and are proportional to the deflection of the slab.

On the basis of these assumptions the following equations for maximum deflection and for critical stress were presented:

Maximum deflections:

$$z_c=\left(1.1-0.88\,\frac{a_1}{l}\right)\frac{P}{kl^2}\text{ -------- }(1)$$

$$z_i=\frac{P}{8kl^2}\text{ -------- }(2)$$

$$z_e=\frac{1}{\sqrt{6}}(1+0.4\,\mu)\frac{P}{kl^2}\text{ -------- }(3)$$

Critical stresses:

$$\sigma_c=\frac{3P}{h^2}\left[1-\left(\frac{Eh^3}{12(1-\mu^2)k}\right)^{-0.15}(a\sqrt{2})^{0.6}\right]\text{ ---- }(4)$$

$$\sigma_i=0.3162\frac{P}{h^2}[\log_{10}(h^3)-4\log_{10}(\sqrt{1.6a^2+h^2}-0.675h)-\log_{10}k+6.478]\text{ -------- }(5)$$

$$\sigma_e=0.572\frac{P}{h^2}[\log_{10}(h^3)-4\log_{10}(\sqrt{1.6a^2+h^2}-0.675h)-\log_{10}k+5.767]\text{ -------- }(6)$$

In the above equations, the following notation is used:

z_c, z_i, z_e, maximum deflection for corner, interior, and edge loadings respectively.

σ_c, σ_i, σ_e, maximum tensile stress for corner, interior, and edge loadings respectively.

P = applied load, in pounds.

h = thickness of slab, in inches.

a = radius, in inches of the circular area (cases I and II) or the semicircular area (case III) over which the load P is assumed to be uniformly distributed.

$a_1=a\sqrt{2}$ in the case of the corner loading, the distance from the extreme slab corner to the center of the area of load application when the slab edges are tangent to that area.

$l=\sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}}$ a dimension, termed the radius of relative stiffness, measured in inches.

k = modulus of subgrade reaction, in pounds inch^{-3}.

E = modulus of elasticity of the concrete, in pounds inch^{-2}.

μ = Poisson's ratio for concrete.

It will be noted that the elastic constants E and μ do not appear in equations (5) and (6). In the original presentation these equations were limited to the case where $E=3{,}000{,}000$ lbs. in.$^{-2}$ and $\mu=0.15$. Subsequently Westergaard generalized these equations and transcribed equation (4). These equations as restated are:

$$\sigma_c=\frac{3P}{h^2}\left[1-\left(\frac{12(1-\mu^2)k}{Eh^3}\right)^{0.15}(a\sqrt{2})^{0.6}\right]\text{ ------ }(7)$$

$$\sigma_i=0.275(1+\mu)\frac{P}{h^2}\log_{10}\left(\frac{Eh^3}{kb^4}\right)\text{ ------ }(8)$$

$$\sigma_e=0.529(1+0.54\mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{Eh^3}{kb^4}\right)-0.71\right]\text{ --- }(9)$$

the term b being an equivalent radius dependent upon a and h and expressed in inches.

In this subsequent paper (*25*) Westergaard proposed a new coefficient, K, defined by the relation

$$K=kl \text{ (measured in lbs. in.}^{-2}\text{).}$$

This new coefficient K, like k, is a measure of the resistance of the subgrade. The reason given for the proposal was the expectation that K would be less dependent on the stiffness of the slab than is k and Westergaard stated his belief that "the truth may lie between the two extremes of a constant k and a constant K."

Westergaard in this subsequent discussion introduced another coefficient D which he called the deflection modulus of the pavement. This was defined as

$$D=kl^2=Kl$$

He restated equations (7), (8) and (9) in terms of the new coefficient of subgrade stiffness and gave a new equation for the maximum stress for the interior case of loading based upon the conception that, for this case, the reactions of the subgrade will be more closely concentrated around the load than are the deflections. The modified equation for maximum stress for the interior case of loading is

$$\sigma''_i=0.275(1+\mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{Eh^3}{kb^4}\right)-54.54\left(\frac{l}{L}\right)^2Z\right]\quad(10)$$

in which,

L is the maximum radial distance from the center of load application within which a redistribution of subgrade reactions is made, in inches.

Z is a ratio of reduction of the maximum deflections which Westergaard states may be expected to vary between 0 and 0.39. If $Z=0$, equation (10) reduces to equation (8).

It is with the equations given above that the discussion which follows will be concerned. For a more complete explanation of the terms used, reference to the Westergaard papers (*23*) and (*25*) is suggested.

EXPERIMENTAL PROCEDURE DESCRIBED

In the original planning of the investigation at Arlington four sections of uniform thickness were provided. Each test section was 20 feet wide by 40 feet long divided into four equal quadrants by a longitudinal joint and a transverse joint. The thicknesses of the four sections of uniform depth were 6, 7, 8 and 9 inches respectively. Except in the case of a few special studies, the test sections were protected in order to maintain them in a uniform moisture condition and to prevent warping from variations in temperature while load testing was in progress. The method used for this protection is described in detail in the first of this series of reports (*18*).

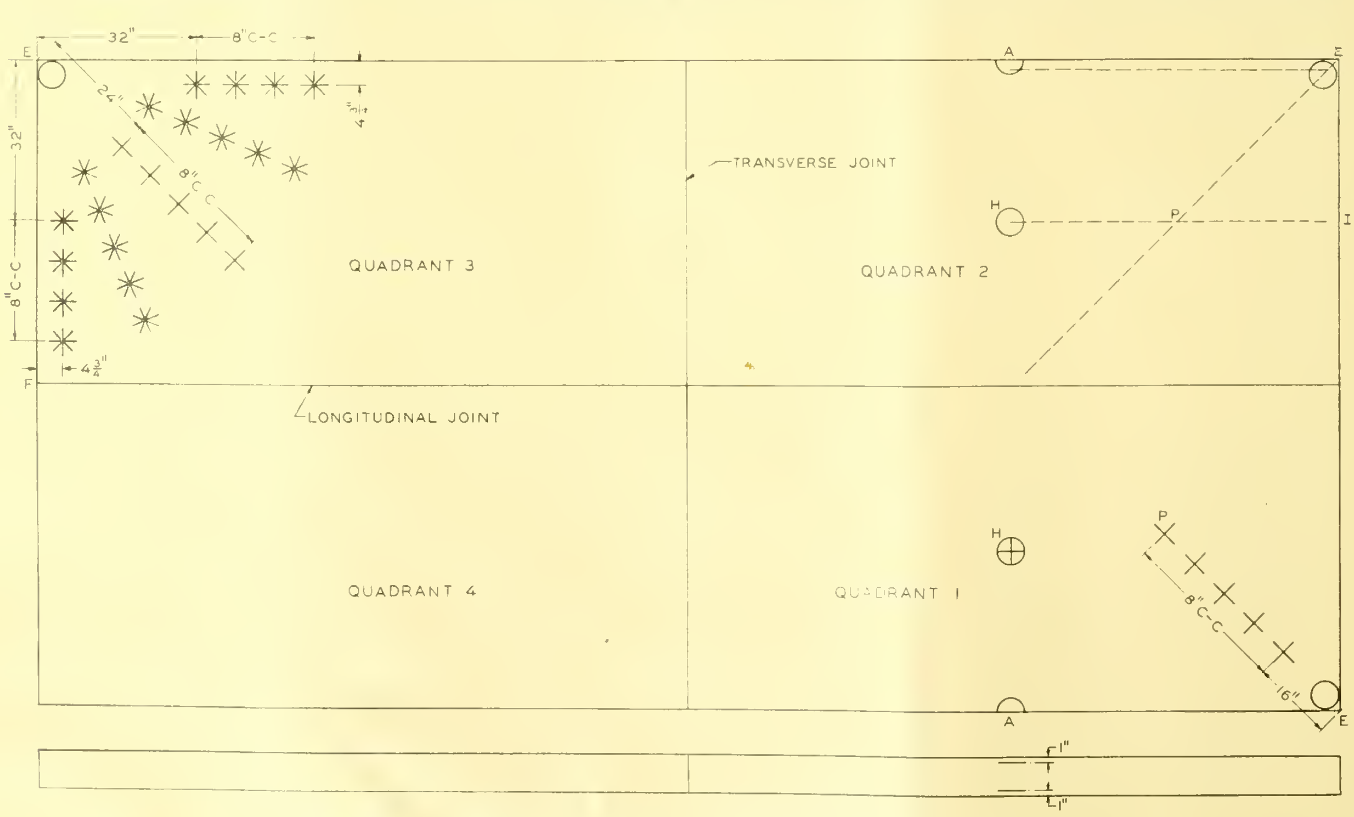
It was stated in the first report that the program of tests for these sections of uniform thickness was planned in such a way that each of the factors which theoretically might influence the load-stress relation could be examined experimentally and the observed effects compared with those predicted by the theory. Thus originally it was planned to study the effect on the load-stress relation of slab thickness, of load position and of size of bearing area. As the work proceeded the study was extended considerably beyond the limits originally contemplated and an examination was made of various assumptions used in the development of the equations and of the several coefficients that appear in them.

In general, loads were applied to the test sections at the three positions assumed in the development of the Westergaard analysis, i. e., the corner, an interior point and a free edge. These positions are shown at E, H, and A on the plan of a typical test section shown in figure 13. The positions of the strain gages installed for each case of loading are shown by the short solid lines in quadrant 1 while the dash lines in quadrant 2 show where the slab deflections were measured.

In certain studies of the structural action of slab corners to be described later, strains were measured in four directions in the corner area. The positions of the strain gages used in this part of the investigation are shown in quadrant 3.

It will be noted that for case I, with the load acting at E, both the strains and the deflections were measured along the bisector of the corner angle E–P. For case II, with the load acting at H, strains were measured in two directions immediately under the load and deflections were measured along the line I–H. For case III, with the load acting at A, the strains were measured on the slab edge 1 inch below the top surface and 1 inch above the bottom surface directly opposite the center of the load A while the deflections were measured along the line E–A.

The apparatus used for obtaining and measuring test loads, the various plates used in applying these loads to the pavement, and the instruments used in



CIRCLES AND SEMI-CIRCLES SHOW POSITIONS AT WHICH LOADS WERE APPLIED.
CROSSES (QUADRANTS 1 AND 3) AND ROSETTES (QUADRANT 3) SHOW STRAIN GAGE POSITIONS.
DASH LINES (QUADRANT 2) SHOW LINES ALONG WHICH DEFLECTIONS WERE MEASURED.

FIGURE 13.—PLAN AND ELEVATION OF A TYPICAL 20- BY 40-FOOT TEST SECTION.

the strain and deflection measurements were completely described in the first report of this series (*18*) and the details will not be repeated here.

The bearing plates used for the corner and interior cases of loading were circular. For the case of edge loading Westergaard assumed the loaded area to be semicircular in shape and in the major part of the tests being reported semicircular plates were used for loads applied at the edge. In certain of the edge tests, however, circular plates were used and in such cases the fact is noted either in the text or on the figures.

The loading tests on sections of uniform thickness considered in this report can be roughly divided into four groups. First, there were a number of what may be called auxiliary or collateral tests designed to settle questions of instrumentation or to supply information needed for the proper execution of the main program or for the interpretation of the data obtained in it. Second, there were certain preliminary or exploratory tests on the experimental sections themselves designed to develop satisfactory methods for obtaining the desired information. Third, there was the main program which comprised the determination of the load-deflection and load-strain relations for the four uniform thickness test sections under the conditions prescribed for the tests together with the determination of values for the several coefficients that appear in the theoretical equations. Fourth, there were a few supplementary studies made because, with test sections available and a testing technique developed, it seemed desirable to obtain data that would throw light on certain problems which were not a part of the original program. If these objectives are kept in mind, it is believed that the purpose for which given tests were made will be clear in the subsequent discussion.

METHOD OF STRESS DETERMINATION OUTLINED

In general the method of arriving at experimental stress values was the same as that described in the earlier reports of this investigation. However, there were some unusual conditions involved in the program being described which made special studies of the strain measuring technique desirable.

It will be recalled that the method used generally throughout this investigation was to measure strains with a temperature compensated recording type of gage (*17*) installed between metal points set in the surface of the pavement slab. From the strains recorded by these gages, stress values were obtained by means of the equations

$$\sigma_x=\frac{E}{1-\mu^2}(e_x+\mu e_y) \text{------------} (11)$$

$$\sigma_y=\frac{E}{1-\mu^2}(e_y+\mu e_x) \text{----- -----} (12)$$

in which,

σ_x is the stress in the direction of the x-axis.
σ_y is the stress in the direction of the y-axis.
e_x is the unit deformation in the direction of the x-axis caused by stress in that direction.
e_y is the unit deformation in the direction of the y-axis caused by stress in that direction.
E is the modulus of elasticity of the concrete.
μ is Poisson's ratio for the concrete.

In applying these equations to measured strains, values for E were determined experimentally because of the importance of the term in the equations. Poisson's ratio, on the other hand, was assumed and the value 0.15 was used in all computations in which the term entered. There were two reasons for this, first, such experimental values of the ratio as are available lie generally between 0.10 and 0.20 with the majority in the upper half of the range and second, small changes in the value of the ratio have an unimportant effect on the computed stress values.

It is obvious that, in any comparison of theoretical stresses with values determined experimentally, it is essential that the strain measuring technique be such as to yield representative values. In this particular investigation the question of gage length was of unusual importance because stress values are the basis for the principal comparison and the test conditions are such as to produce rather abrupt stress variations in the area where strain measurements were to be made. For theoretical reasons, therefore, a short gage length is indicated. Experience has shown, however, that because concrete in small masses may lack the homogeneity that is assumed, the use of too short a gage length is likely to cause trouble by giving strain values that are not representative. A short gage mounted directly over a large piece of aggregate may show quite a different strain indication than would a similar gage mounted between two such pieces because the modulus of elasticity of the stone is different from that of the mortar. A fairly long gage tends to average these deformations and to indicate unit strains which, when combined with the "average" modulus of elasticity, will give stress values that are nearly correct.

In this investigation a collateral study was made to determine the length of gage necessary to give representative strain values in the tests that were scheduled. It was found that a gage length of about 6 inches was sufficient to average out local effects of aggregate in the concrete being used. This was evidenced by the fact that when this gage length was used a given load applied at homologous points on a given test slab always produced essentially the same maximum strain.

Another study was made to determine the effect of gage length on indicated unit strains when the measurements were being made within the area over which the tests loads were applied. For this study a special gage having an effective gage length of 2.2 inches was constructed and this gage was used to determine strain variation under the bearing plates of 6-, 8-, 12-, 16- and 20-inch diameters. Figure 14 shows typical data obtained in these tests. The curve marked A shows the maximum unit strain values recorded in a 2.2-inch gage length at the middle of a diameter of the bearing area for a given applied load and each size of bearing area listed above. Curve B shows, for the same load and bearing areas, unit strain values which are averaged from three 2.2-inch gage lengths arranged end to end along one diameter of the bearing area. These average unit strains, therefore, are those which would be shown by a single 6.6-inch gage centrally located along the same diameter. The location of the gage lengths with respect to the bearing area is shown in the legend of figure 14.

Because of possible local aggregate effects with a gage of this length the actual procedure was to leave the gage in one position and to shift the position of the loaded areas with respect to the gage length. The data indicate that for the conditions of these tests, the stress variation along a diameter of a bearing area was not sufficient to cause different unit strains for gage lengths

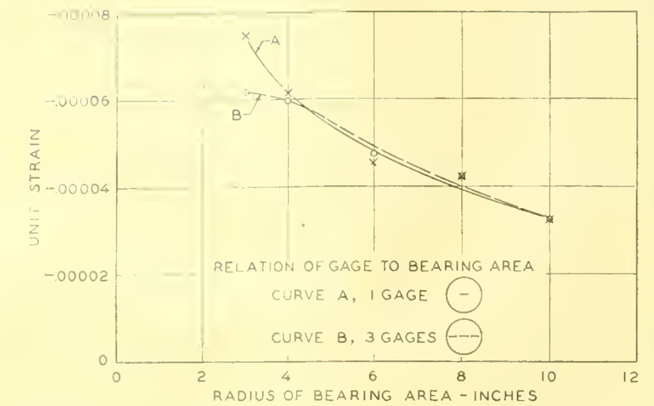


Figure 14.—Comparison of Average Unit Strains Measured Within Bearing Areas of Different Sizes. The Strains Were Measured Over A Distance of 2.2 Inches for Curve A and Over a Distance of 6.6 Inches for Curve B.

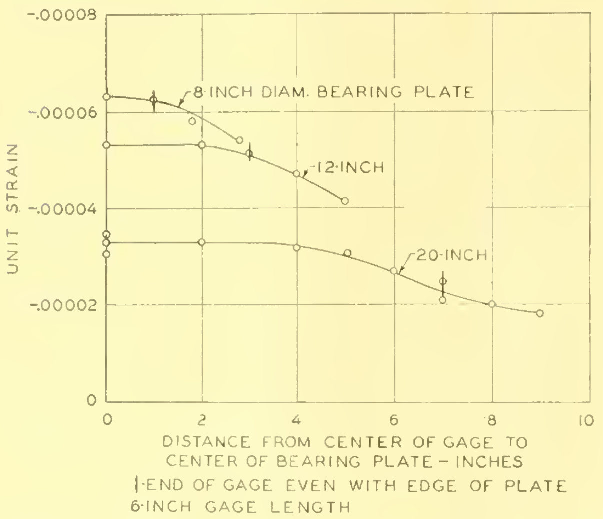


Figure 15.—Variation in the Magnitude of Indicated Unit Strain Under Bearing Plates of Three Sizes.

of 2.2 inches and 6.6 inches respectively so long as the plate diameter was 8 inches or greater and the gage was positioned along a diameter of the area.

In this same group of tests some study was made of the strain variations within the bearing areas when a gage length of about 6 inches was employed. Bearing areas of 8-, 12- and 20-inch diameters were used and the tests were made by shifting the bearing area with respect to the gage length in the manner previously described. However, in these tests, the strain determinations were made over the entire length of the diameter in each case. The data obtained are shown in figure 15. It appears from these data that the maximum unit strain occurs under the center of the bearing area, as would be expected, and that for bearing areas of these sizes there is no appreciable reduction in the strain for some distance toward the edge of the bearing area. In other words the strain variation curve is rather flat in this region. The reduction in strain magnitude near the edge of the bearing area is evident in these data.

In tests such as those just described the precision with which unit strain values are measured is a very important matter. In the published description of the strain gage used so extensively in this general investigation (*17*) it was stated that the accuracy of the gage is sufficient to permit the determination of stress in concrete to within 20 or 25 pounds per square inch, where dependence is placed upon a single observation. Subsequent extensive use of these gages has shown this to be a very conservative estimate. Stress determination is affected by precision in determining the effective modulus of elasticity as well as by precision in measuring strains in the material so that any lack of precision in stress determination may be the combined result of several influences. In this general investigation every effort was made to keep the gages functioning at maximum efficiency and usually dependence was not placed on a single observation unless that observation was supported by other data. In order to find out what consistency could be expected in the load-strain measurements when the same operation was repeated, an 8,000-pound load was applied 80 times at the edge of the 7-inch uniform-thickness test section and the strain recorded for each application. Of the 80 individual determinations only one differed from the average by more than 3½ percent, and some idea of the consistency of the combined load and strain measuring operation may be had from the following figures.

Edge loading, 8,000-pound load, 7-inch slab thickness

Average unit strain	0.00006675
Standard deviation	.00000146
Coefficient of variation 2.18.	

DETERMINATIONS MADE OF MODULUS OF ELASTICITY OF THE CONCRETE

Two methods were used for studying the values of the modulus of elasticity of the concrete, first, the testing of laboratory specimens in compression or bending and, second, by deflection measurements on the test sections themselves.

Stress determinations from measured strains require a knowledge of the value of the modulus and since the exposed pavement sections were tested over a long period of time it was necessary to know also how much the value varied from summer to winter. It was shown in the second report of this series (*19*) that there is an annual change in the moisture state of the concrete in the test sections sufficient to cause rather large volume changes from summer to winter. It is well established that changes in moisture state cause changes in the stiffness of concrete. It seemed probable, therefore, that this change in moisture content from its lowest value in summer to a higher value in winter was such as to cause changes in the modulus of elasticity that would have to be taken into account.

When the test sections were originally laid, extra slabs were provided from which laboratory test specimens could be taken. From these slabs cores were drilled and prisms for flexure testing were sawed and used in laboratory studies of the modulus of elasticity. It was found by flexure tests on prisms that those dried for 12 months by storage in the normal atmosphere of the laboratory had an average modulus of elasticity of 4,500,000 pounds per square inch while a comparable group that had been immersed in water at laboratory temperature had an average modulus of 6,000,000 pounds per square inch.

An attempt was made to determine the moisture content of the concrete in the test pavement in order that some comparison might be made between it and the mositure content of the concrete in the test prisms as an indirect measure of the modulus of elasticity of the concrete in the pavement sections. Fragments broken from the extra slabs on the subgrade were weighed and dried. The effort was not entirely satisfactory, however, because of variation in moisture content from top to bottom of the pavement and because of difficulty in getting a representative sample for the moisture determination. As nearly as could be determined the moisture content of the concrete that had been immersed for 10 months was slightly greater than that of the pavement during the wet winter months. It seems reasonable to conclude, therefore, that the modulus of elasticity of the pavement concrete in winter would be somewhat less than 6,000,000 pounds per square inch. While it was not possible to establish a reliable comparison between the moisture content of the air dry prisms and that of the pavement during the dry summer condition it is believed that the difference in the corresponding modulus values would not be large and that the value of 4,500,000 pounds per square inch determined by the laboratory tests might be used in conjunction with the strains measured during the summer tests.

Values for the modulus of elasticity determined from slab deflections will be discussed later and comparisons will be made between them and the values determined from the flexure tests of prisms.

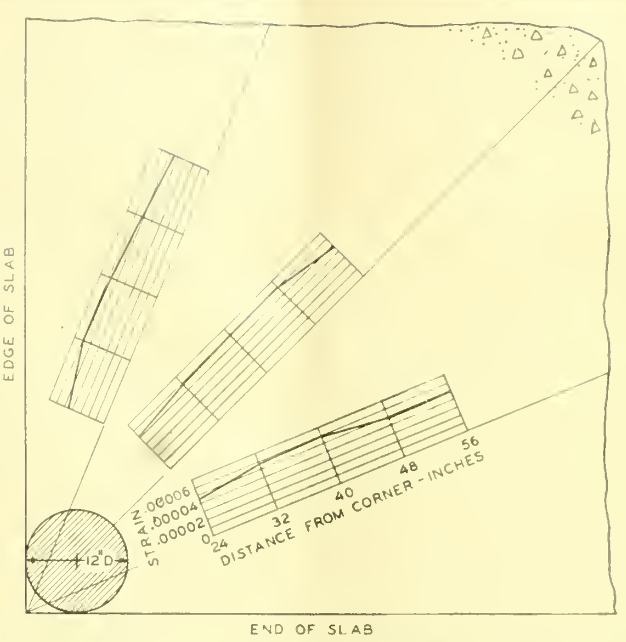


FIGURE 16.—VARIATION IN MAGNITUDE OF MAXIMUM STRAINS—7-INCH UNIFORM-THICKNESS SECTION—SYMMETRICAL CORNER LOADING.

POSITION AND DIRECTION OF CRITICAL STRESSES INVESTIGATED

Before attempting to make direct comparisons between computed and observed stresses it was considered desirable to make certain exploratory tests on the test sections to determine the position and direction of the critical stress for each position of loading, to ascertain the effect of repetition of load on stress magnitude and to establish the relation between load magnitude and stress magnitude for each case.

In the discussion of the comparisons of computed and observed deformations which is presented later it was found convenient to discuss the cases of corner, interior and edge loading separately and in that order. Because of this fact, the discussion of the work that comprised the preliminary load tests has been arranged in the same order.

A study was made of the strain variations in the corner area of a typical section for case I (the corner loading) in order to determine the location and direction of maximum strain. Strains were measured in two directions at various positions along the bisector of the corner angle and in four directions at comparable positions along two other lines radiating from the slab corner and making an angle of 22.5° with the corner bisector. The maximum strain at each position along the three radial lines for the particular test conditions that obtained is shown in figure 16. Along the bisector of the corner angle the direction of the maximum stress was known to be parallel to the bisector but along the other two radial lines the direction of the maximum strain was found to make an angle of about 15° with the radial line in each case. This matter of the direction of stresses in the corner area will be discussed much more fully later in the report.

The strains in the vicinity of the slab edges are not shown in figure 16 but it has been well established that for a corner loading on a concrete pavement slab of uniform thickness the strains along the edges that form the corner are definitely of less magnitude than those along and in proximity to the corner bisector. The data shown in figure 16 are typical of those obtained in a number of similar corner tests and these data consistently indicated that the strain measured along the corner bisector at the proper location would be at least as large as, if not larger than any other strain to be found in the corner region. This being true it is only necessary to locate the position along the bisector at which the radial strain reaches a maximum and at this position measure the radial strain and the strain at right angles to it. The combination of these two strains determines the critical stress for the case of the corner loading.

In his analysis of this case Westergaard gives an equation for finding the distance from the corner along the corner bisector at which the maximum stress theoretically develops. This equation is

$$x_1 = 2\sqrt{a_1 l}$$

in which

x_1 is the distance from the corner to the point of maximum stress measured along the bisector of the corner angle.

a_1 and l are as previously defined.

The distance x_1 was determined experimentally for each of the slab thicknesses, for a range in loads, after various numbers of load applications and for different conditions of temperature warping.

In table 2 the experimentally determined values of this dimension are given for the four slab thicknesses, a range of loads and after various numbers of loadings had been applied. The tests covered a considerable period of time so that the soil moisture was not the same at the time the different sections were tested. For example, when the 6-inch and 7-inch sections were tested the subgrade soil was in a relatively dry state and a slight separation between the lower surface of the slab was noted after the application of a number of loads. In contrast, the subgrade was in a wet condition after severe freezing and thawing when the comparable tests were made on the 8-inch and 9-inch sections.

TABLE 2.—*Experimentally determined values of the distance, x_1, along the bisector of the corner angle at which the maximum stress was found to occur. These values are averages from the stress curves of the four quadrants. Loads were applied on 12-inch diameter bearing plates*

Slab thickness	Load	Distance, x_1		
		After first load	After 12 load applications	After 35 load applications
Inches	*Pounds*	*Inches*	*Inches*	*Inches*
6	4,000	34	35	35
	5,000	34	33	35
	6,000	33	33	33
7	4,000	35	34	38
	5,000	35	34	37
	7,000	33	33	34
	9,000	33	34	33
8	3,000	33	43	41
	6,000	33	33	39
	8,000	33	33	32
	10,000	33	33	31
9	3,000	46	48	46
	6,000	41	47	43
	8,000	41	40	42
	10,000	41	39	40

The data of table 2 indicate a tendency for the distance x_1 to increase, for the low loads, with the number of load applications. For the higher loads the distance remained fairly constant as the loads were repeated. There seems also to be a tendency for the distance to decrease as the magnitude of the load is increased, other conditions remaining the same.

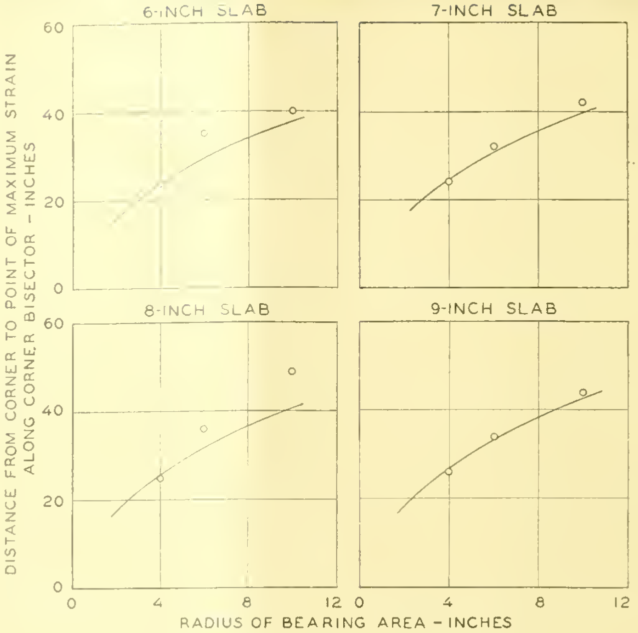
In table 3 are shown experimentally determined values for x_1 for three conditions of temperature warping. Each value in this table was from an average stress curve, each point of which was the average of eight separate stress determinations. The values in table 3 tend to decrease as the degree of contact between the slab corners and the subgrade is increased. This same effect was noted in the data of table 2 as the magnitude of the load was increased.

TABLE 3.—*Experimentally determined values of the distance, x_1, along the bisector of the corner angle at which the maximum stress was found to occur. These values are averages from the stress curves of the four quadrants for each condition of warping. Loads were applied on 12-inch diameter bearing plates*

Slab thickness	Distance, x_1		
	Edges warped down	Slab flat	Edges warped up
Inches	*Inches*	*Inches*	*Inches*
6	31	36	37
7	30	34	36
9	31	33	38

With the possible exception of those for the 9-inch slab the experimentally determined values for x_1 were found to be in good agreement with computed values. This is especially true for the condition of the unwarped or the downward warped slab corners. When the corner was warped upward an increase in the value of the distance x_1 was noted.

In figure 17 average values of x_1 from a considerable number of tests with the pavement in an unwarped condition are compared with values computed with the equation mentioned above. It will be noted that the comparisons include three sizes of bearing area and four slab thicknesses.



CURVES SHOW EQUATION $x_1 = 2\sqrt{a_1 \ell}$ IN WHICH ℓ WAS DETERMINED BY MEASURED DEFLECTIONS.

○-VALUES OF x_1 FROM STRAIN MEASUREMENTS.

FIGURE 17.—COMPARISON OF THE OBSERVED AND THEORETICAL DISTANCES FROM THE CORNER TO THE POINT OF MAXIMUM TENSILE STRAIN ALONG THE BISECTOR OF THE CORNER ANGLE.

Generally there is good agreement between the computed and observed values of this dimension. While there is some tendency for the experimentally determined values to be greater than the computed values, particularly with the larger bearing areas, the difference is not great and when the flat character of the stress variation curve along the corner bisector is considered, it is apparent that small errors in determining the value of x_1 would be of very little importance. It is concluded, therefore, that the theoretical equation for computing this distance is sufficiently accurate for all practical purposes.

EFFECTS OF REPEATED CORNER LOADING STUDIED

The behavior of a slab corner under repeated loading was first studied in a series of tests on the 7-inch uniform-depth test section made during the late fall when the subgrade soil was in a normal state. A load of 8,000 pounds was applied 80 times to the free corner of each of the four quadrants of the test section. Each load was maintained at full value for 1 minute and after the removal of the load 1 minute was allowed to elapse before the next loading was begun. The loads were applied in groups of ten and the maximum strains for each group of 10 loadings were averaged. The data obtained are shown in figure 18. Since, in this graph the data from all four quadrants are combined, each point on the curve is an average of 10 tests in each of four quadrants or 40 observed values.

The data show a distinct tendency for the strain to increase with repetition of the test load, the average strain for the last group of ten loadings being approximately 10 percent greater than the average strain for the first group. The rate of increase is not constant, decreasing as the number of load applications increased. The increase after 60 repetitions of the load was very small.

Following these experiments it was decided to investigate the relation for all four thicknesses of pavement. In carrying out this subsequent program the procedure followed with each test section was the same and comprised the following schedule:

1. Two applications of a test load of each of three or four different magnitudes were made on each of the four free corners of each of the four sections. For each load application the maximum deflection and the maximum strain were measured.
2. A series of 12 loads of the maximum magnitude used in (1), above, was applied but no measurements of deflection or strain were made.
3. The same loadings and measurements made under (1), above, were repeated.
4. A series of 50 to 70 loadings of the maximum magnitude used in (1), (above), was applied, no measurements of deflection or strain being made.
5. The same loadings and measurements made under (1), above, were again repeated.

The load-deflection and load-strain relations developed during the tests of this schedule are shown in figure 19 while the strain variation along the bisector of the corner angle, for the maximum load magnitude is shown for each slab thickness in figure 20. Each point shown in these two figures is an average of two measurements in each of four quadrants or eight observations in all.

The tests extended over a winter period so that the subgrade was not in the same condition when all of the sections were tested.

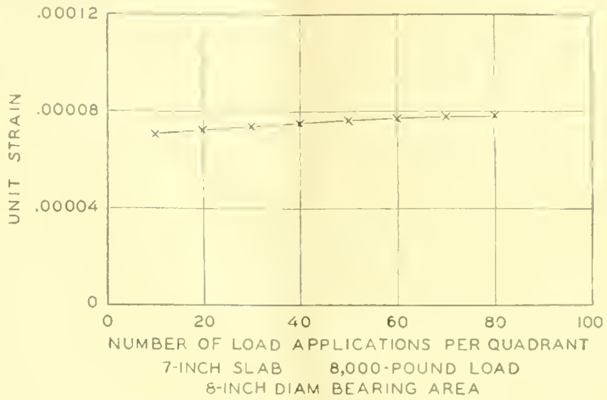


Figure 18.—Effect on Strain of Repeated Loads Applied at the Corner of a Concrete Pavement.

The 7-inch section was tested in early December a short time after the data shown in figure 18 were obtained. The earlier series of tests may have affected the soil under the slab corners. The 6-inch section was tested in early January. Although the soil beneath the pavement was not frozen when the tests were made it probably had been frozen slightly a short time before. The tests on the 8-inch section were made during March shortly after the subgrade had thawed after having been frozen to a depth of about 12 inches. During the time that the soil was frozen the test section was heaved upward approximately 1 inch but at the time of load testing it had resumed approximately its original position. The soil was in a wet condition but there is reason to believe that the pavement was in good contact with the soil over its entire area when the load tests were made. The 9-inch section was tested about the first of April. The subgrade had been frozen

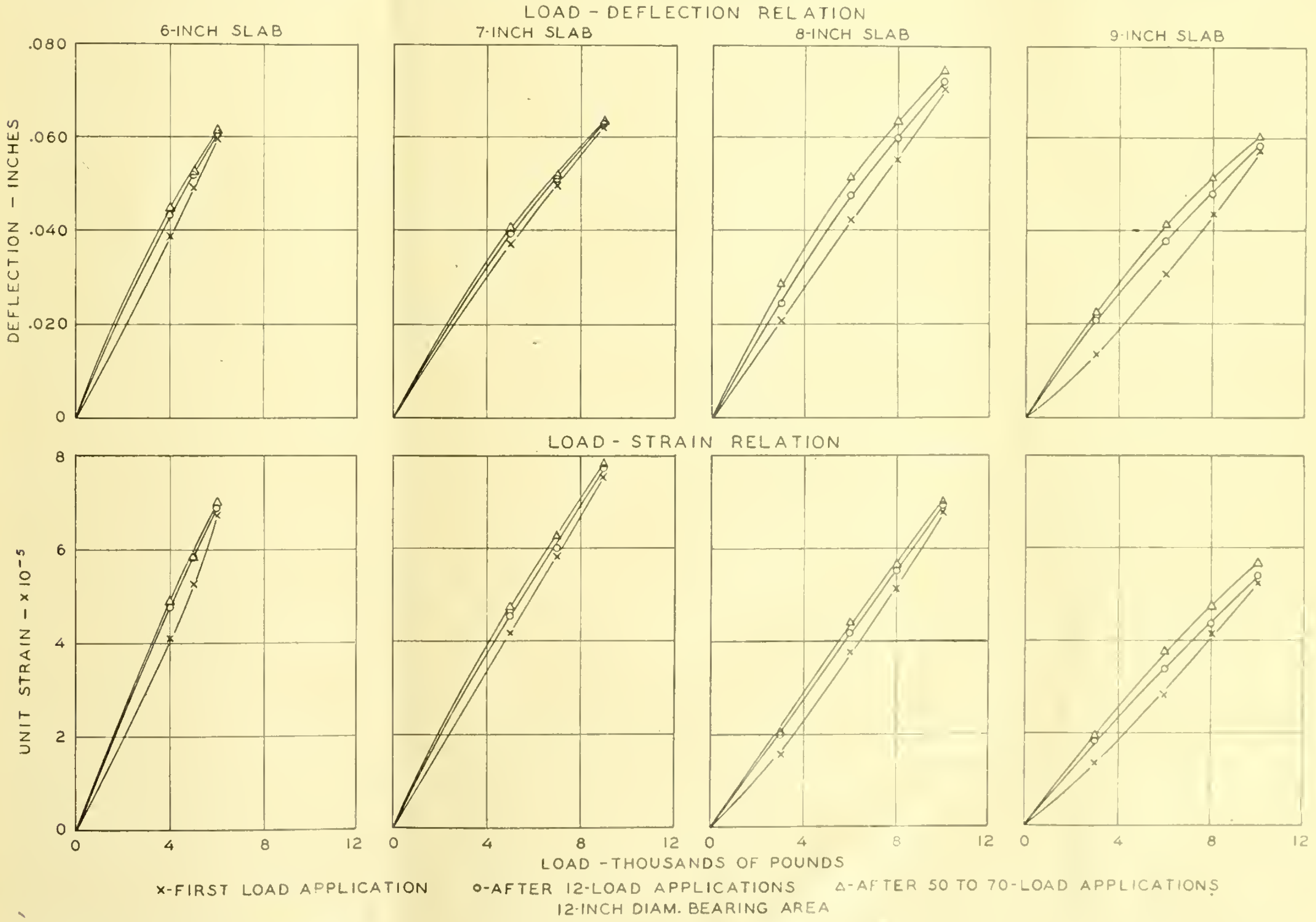


Figure 19.—Effect of Repeated Loading on the Load-Deflection and Load-Strain Relations at a Free Corner

as described above and, in addition, the section had been inundated for about 30 hours by flood backwater. The soil was, therefore, quite soft at the time the tests were made on this section. The swelling of the soil from moisture probably improved the uniformity of its contact with the lower surface of the pavement, however.

It is indicated by the data of figure 19 that the deflections and the stresses caused by a given load increased with the repetition of the load for all thicknesses in the same manner as found earlier with the 7-inch section. It will be noted also that when the load magnitude was greatest the effect of repeated loading on deflection and stress was least. The data consistently indicate that the change in deflection and strain under repeated loading is the result of an adjustment of the conditions of soil support under the slab corner as the repetition of loading proceeds. This adjustment continues until a condition of equilibrium is established.

The lack of linearity in the load-deflection and load-strain data for these slab corners is further evidence that as the slab corner is deflected the conditions of soil reaction under it change.

Similar tests to determine the load-deflection and load-strain characteristics of the interior region of the test sections were made and representative data from these are shown in figures 21, 22 and 23.

Figure 21 is a typical strain variation graph for an interior loading. Each point on the curves is the average of two strain measurements in each of four quadrants, eight observations in all. The radial strains were measured along a radius from the center of the loaded area and at distances from that center as shown. The tangential strains were measured at the same positions but in a direction normal to the radius. The shape of the curves under the bearing plate was drawn in accordance with data discussed earlier in this report and shown in figure 15.

These data indicate that directly under the bearing area the two strains have essentially the same magnitude but at points not within the bearing area the tangential strain is greater than the radial strain. The radial strains become zero at a distance from the center of the bearing area approximately equal to *l* while the tangential strains approach zero at a distance of about 1.6*l*. The data in figure 21 were obtained on the section of 9-inch thickness but tests on the sections of other thicknesses showed the relation to be essentially the same when expressed in terms of *l*. This means that the critical stress from a given wheel load will not be increased by the presence of another equal load so long as the two loads are separated by a distance of 1.6*l* or more. For most highway pavements this would correspond to a distance of 40 to 60 inches.

It is interesting to note how closely this is in accord with experimental strain data obtained in studies of the effect of 6-wheel trucks on concrete pavements made by the Public Roads Administration some 17 years ago (*16*).

PRELIMINARY TESTS YIELDED SIGNIFICANT RESULTS

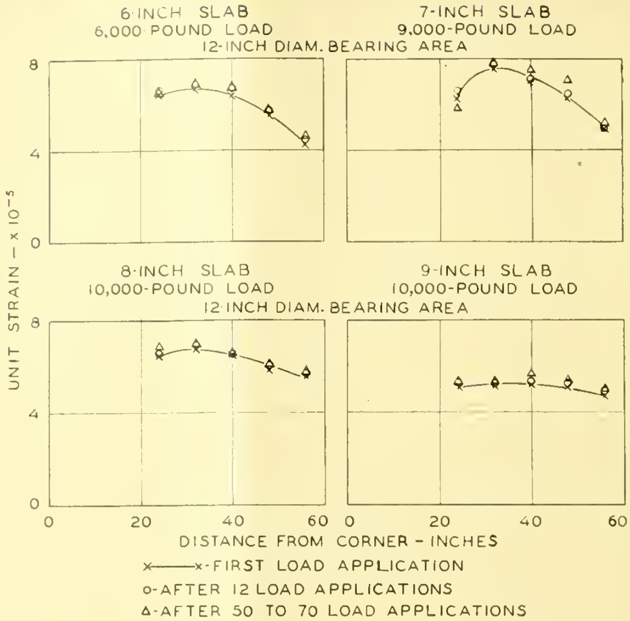


Figure 20.—Effect of Repeated Loading on the Strain Distribution Along the Bisector of the Corner Angle.

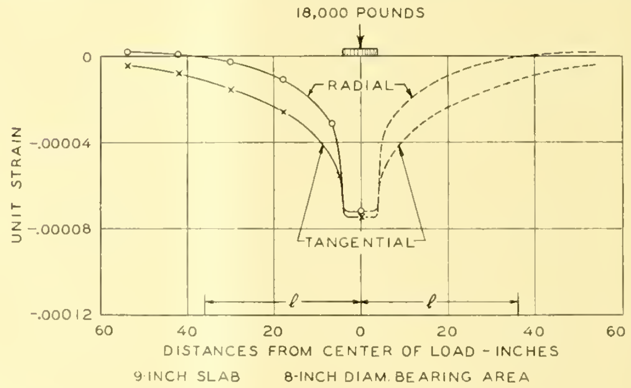


Figure 21.—Variation in Tangential and Radial Strains in the Vicinity of a Loaded Area at the Interior of a Concrete Pavement Slab.

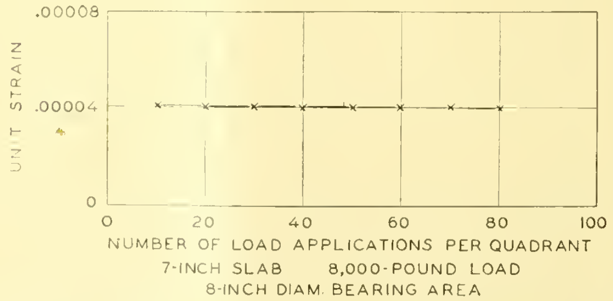


Figure 22.—Effect on Strain of Repeated Loads Applied at the Interior of a Concrete Pavement.

Repeated load tests were made at the interior position (case II) following the same procedure as that used for the corner case. Typical data, obtained in the tests on the 7-inch thickness section, are shown in figure 22. The data from the tests at the interior consistently show that repeated loading causes no increase in the magnitude of the critical strain, within the limits of the tests of this program.

A large number of tests were made in studies of the load-deflection and load-strain relations for the interior loading. Typical data from these tests are given in figure 23. Both the load-deflection and the load-strain relation were consistently linear for the interior loading, within deflection and strain limits such as those shown in this figure.

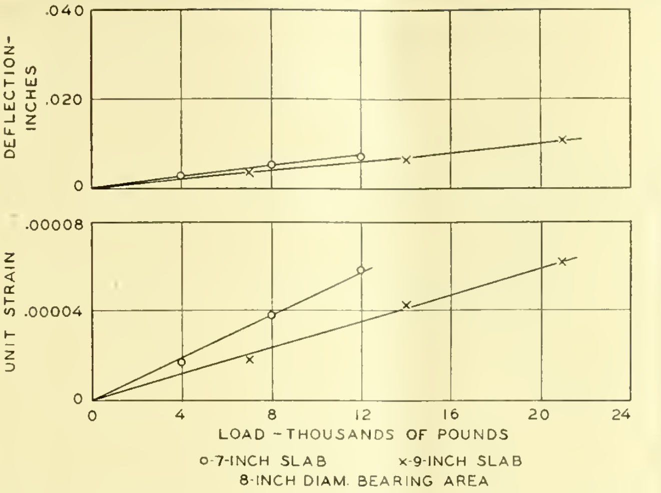


Figure 23.—Typical Load-Deflection and Load-Strain Relations at the Interior of a Concrete Pavement.

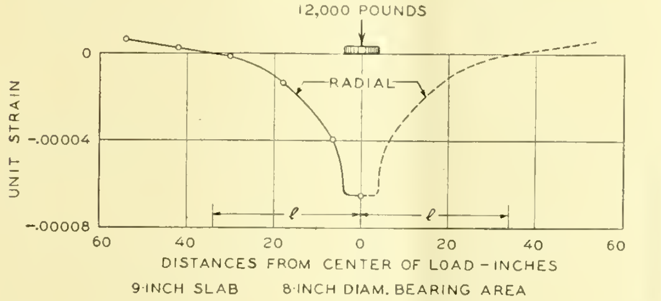


Figure 24.—Variation in Radial Strains Along the Edge of a Pavement for a Load Applied on a Circular Area at the Edge.

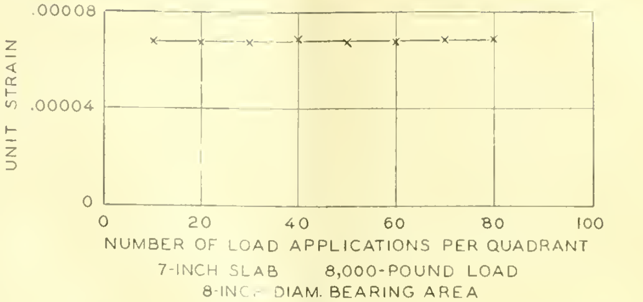


Figure 25.—Effect on Strain of Repeated Loads Applied at the Edge of a Concrete Pavement.

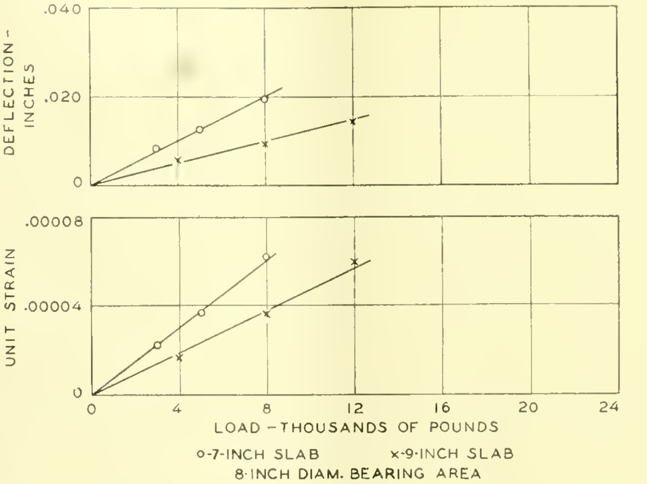


Figure 26.—Typical Load-Deflection and Load-Strain Relations at the Edge of a Concrete Pavement.

At the free edge, tests were made to determine the shape of the strain variation curve along a line through the center of the bearing area and parallel to the edge of the pavement; to determine the effect of repeated loads; and, to determine the load-deflection and load-strain relations using circular bearing areas centered 6 inches from the slab edge. Typical data from these tests are shown in figures 24, 25 and 26.

The strain variation curve (fig. 24) shows radial strains only since tangential strains at the points of measurement shown are negligible. Each point shown on the curve is an average of eight observations, two in each of the four quadrants of the test section. The measurements of strain were made on the upper surface of the slab, hence the maximum strain shown is a compression in the upper surface of the pavement. It will be noted that the distance from the center of the loaded area to the point where the strain changes from compression to tension is approximately equal to *l*. The tensile strain in the upper surface of the pavement is much smaller than that in the bottom surface directly under the area of load application. It is this maximum tensile deformation in the bottom of the slab directly under the load that is the critical strain. Numerous tests have shown that the maximum compressive strain measured under the center of a circular bearing plate of 8- to 12-inch diameter when the plate is tangent to the slab edge is essentially equal to the maximum tensile strain measured at the bottom of the slab edge. In other words the maximum strain shown in the variation diagram may be considered as equal to the critical strain.

The data from repeated loading are given in figure 25. The procedure was the same as that followed for case I (corner) and case II (interior). It is indicated that the critical strain for the edge loading is not increased by a repetition of load application. In this respect it is like the interior case. This difference in behavior between the corner on the one hand and the interior and edge on the other is attributed to two factors, first, there is more deflection and hence more soil deformation when the free corner of a slab of uniform thickness is loaded and, second, for corner loading there is less confinement for the soil than is present with either the edge or interior loadings.

In figure 26 are given some typical load-deflection and load-strain data for the edge loading. It was found that these relations were linear within the limits of the tests.

To summarize, the preliminary load testing program gave data as to the strain distribution which located the position of the critical strain for each of the three cases of loading considered in the Westergaard analysis. It verified the equation for locating the position of maximum stress for corner loads as given by Westergaard. The application of repeated loads was found to affect the magnitude of the critical strain to a measurable extent with corner loading but not for the interior or the edge loadings. Finally the preliminary load testing program showed that the load-deflection and load-strain relations were nearly linear for corner loading and linear for the interior and edge loadings, within the limits of the tests.

LOAD-DEFLECTION DATA AND THEIR SIGNIFICANCE

The methods used for determining the deflected shape of the test sections under various load and other conditions were described and discussed in the earlier reports of this series.

The principal use made of the deflection data in the present study is for evaluating the constants that appear in the various equations of the Westergaard analysis.

In connection with the presentation of his generalized equations (*25*) Westergaard suggests methods for determining the value of the quantities D, l, k, K, E, L and Z from observed values of deflections and stresses. These will be discussed briefly.

The deflection modulus, D, was defined as being equal to kl^2 and values for it may be obtained by substituting the observed maximum deflection value in the appropriate equation for maximum deflection (1), (2) or (3).

The value of l is obtained by adjusting the scales (both horizontal and vertical) of a theoretical deflection diagram, in which the ordinates are deflection units and the abscissas are distances expressed in terms of l, until the theoretical elastic curve coincides as nearly as possible with the observed shape of the deflected slab.

For the corner loading the theoretical deflection diagram may be constructed, once D is determined, by means of the generalized equation for corner deflection

$$z=\frac{P}{D}\left(1.1e^{-\frac{x}{l}}-\frac{a_1}{l}0.88e^{-\frac{2x}{l}}\right) \qquad (13)$$

z being the deflection at any distance x from the slab corner, measured along the bisector of the corner angle, while e is the Naperian base. Values of l are assumed until coincidence is obtained.

For the interior and edge cases, the theoretical deflection diagrams were constructed by means of coefficients of deflection obtained from the diagrams given by Westergaard in the original analysis (*23*) (figures 4 and 8). Values of l were assumed as in the case of the corner loading until the best degree of coincidence was attained.

With values for D and l determined in the manner just described k, K, and E may be evaluated by means of the following equations:

$$k=\frac{D}{l^2} \qquad (14)$$

$$K=\frac{D}{l} \qquad (15)$$

$$E=\frac{12(1-\mu^2)Dl^2}{h^3} \qquad (16)$$

There appears to be no direct method for the determination of values for the quantities L and Z from deflection data. Westergaard suggests that these terms be evaluated from the theoretical equations for stress. Discussion of this question will accordingly be deferred until after the introduction of the stress data. Except where stated otherwise, the value of Z will be taken as zero, however.

Observed deflection data for each of the four test sections of uniform thickness for the three cases of corner, interior and edge loading are shown in figures 27, 28, and 29, respectively. On these diagrams the observed shape of the deflected slab is shown by curves drawn through crosses. Each cross is the average of 12 observed values, three measurements in each of four quadrants. The small circles are computed values which indicate the position of the matched theoretical diagram constructed for the purpose of determing l.

Values of the several coefficients, derived by the methods just described from the deflection data shown in figures 27 to 29, inclusive, are given in table 4.

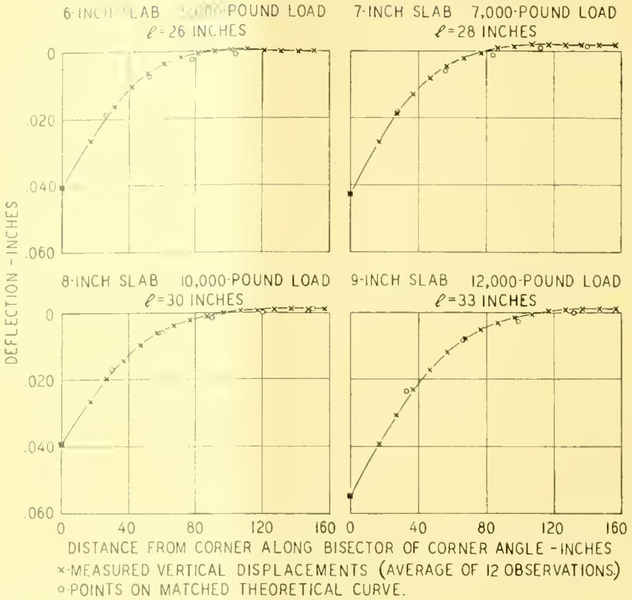


Figure 27.—Deflection of the Bisector of the Corner Angle Used for Evaluating l.

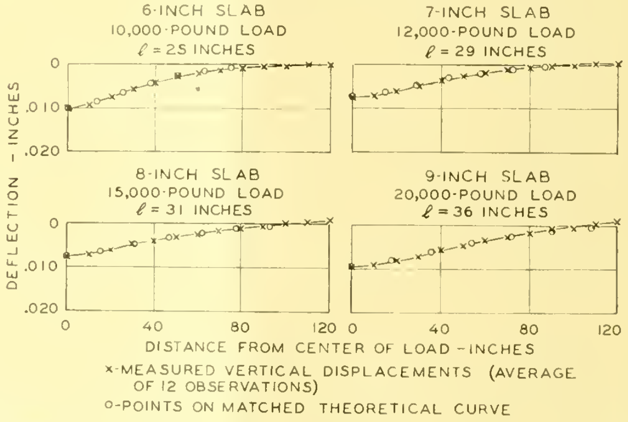


Figure 28.—Deflection of Slab From Interior Loading Used for Evaluating l.

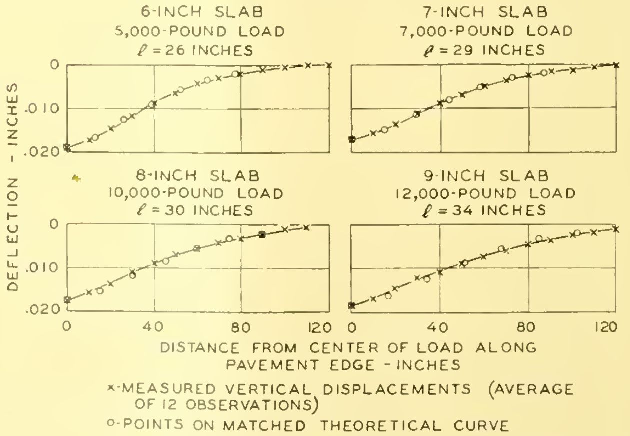


Figure 29.—Deflections Along Free Edge of Pavement Used for Evaluating l.

Table 4.—*Values for various coefficients, used in the Westergaard equations, determined from measured deflections*

Position of load	Time of testing	Slab thickness	l	k	K	D	E
		Inches	*Inches*	*Lbs. in.*$^{-3}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-1}$	*Lbs. in.*$^{-2}$
Corner	Late summer	6	26	143	3, 708	96, 400	3, 540, 000
	Winter	7	28	161	4, 515	126, 400	3, 390, 000
	Winter	8	30	227	6, 825	204, 700	4, 220, 000
	Late fall	9	33	168	5, 535	182, 600	3, 200, 000
Interior	Late summer	6	25	195	4, 880	122, 000	4, 140, 000
	Winter	7	29	238	6, 895	200, 000	5, 750, 000
	Summer	7	28	222	6, 230	174, 400	4, 670, 000
	Winter	8	31	260	8, 065	250, 000	5, 500, 000
	Late fall	9	36	203	7, 315	263, 200	5, 490, 000
	Summer	9	33	220	7, 290	240, 500	4, 210, 000
Edge	Late summer	6	26	171	4, 440	115, 400	4, 235, 000
	Winter	7	29	212	6, 145	178, 200	5, 125, 000
	Winter	8	30	279	8, 365	251, 000	5, 175, 000
	Late fall	9	34	243	8, 260	280, 800	5, 220, 000

The deflection tests on the four sections were not made at the same time but covered a period of about 4 months, as follows:

Test section:	*Month tested*
6-inch	September.
7-inch	December.
8-inch	November.
9-inch	October.

During the period when a particular section was under test it was insulated against air temperature fluctuations with a layer of straw and was protected from precipitation and direct sunlight with a temporary canvas shelter. However, during the 4-month period between late summer and winter there are important changes in temperature, evaporation rate, ground moisture, etc., and it seems reasonable that some effects of this seasonal change are present in the data. If so, the effect would be most noticeable in comparisons between different test sections.

For a given slab thickness, values of the radius of relative stiffness, l, are in good agreement for the three cases of loading. For conditions that are comparable there is rather good agreement also between the values of modulus of subgrade reaction, k, as determined by pavement deflection, for the interior and edge loadings but the value for the corner loading is consistently lower. This is believed to be the result of incomplete contact between the corner area and the subgrade and is in accord with the evidence of the strain data previously discussed. It will be noted that the values of k, as determined from the deflections of the 6-inch section of uniform thickness are somewhat lower than as determined by the deflection data from the three other test sections.

SEASONAL CHANGES AFFECT CERTAIN COEFFICIENTS

In the earlier discussion of the modulus of subgrade reaction, k, in the first section of this report, the data from the bearing plate tests of series 4 showed that for a given plate displacement the value of k was appreciably greater in summer than in winter. For example, at a displacement of 0.02 inch, the summer value was 280 lbs. in.$^{-3}$ and the winter value was 199 lbs. in^{-3}. This same trend is not evident in the limited data on the values of k, determined from the deflections of the interior of the 7-inch and 9-inch sections shown in table 4. It will be noted that the value of k is nearly the same for summer conditions as it is for late fall or winter conditions in the two comparisons available in this table. It may be recalled that in the study of seasonal effects on the value of k with the bearing plate tests, the plate was left continuously in place on the subgrade. While the plate used was relatively large, it actually was much smaller than a full-size pavement slab. It is conceivable, therefore, that a larger change in soil moisture may have occurred under the bearing plate than was possible under the test section and that this larger change in soil moisture might account for the larger variation in the value of k as determined from the bearing plate test.

The general level of values of k from the bearing plate tests is in reasonably good agreement with that determined from pavement deflections.

The coefficients D and K being dependent on l and k are affected directly by any conditions that influence the value of either l or k. The values of the modulus of elasticity for the concrete, E, as determined from the slab deflections are in the same general range as the values that were obtained from the tests of the laboratory specimens. Values obtained from corner deflections are distinctly lower than those determined by deflection data from interior or edge loading. This is believed to be a direct reflection of the imperfect subgrade support that obtained under the unwarped slab corners and of certain rotational movements that will be discussed presently. For this reason the value of E, determined from corner deflections, is considered a less reliable index of slab stiffness than those determined from interior or edge deflection data. It is noted also that there is a tendency for the values determined from the deflections of the 6-inch section to be slightly lower than those for the other three sections. A comparison of the data obtained in summer with those obtained in winter in tests at the interior of the 7-inch section show a much higher value of E for the winter condition. The same trend is evident in the tests made in summer and in late fall at the interior of the 9-inch section.

Since the value of the modulus of elasticity of the concrete varies with the moisture state of the material and since the moisture state is known to vary but no good measure of its range or rate of variation was available, an uncertainty as to the exact value of the modulus E was always present in any consideration of data that required its use, particularly where long time periods were involved in the testing.

Assuming that values of $E=5{,}500{,}000$ lbs. in.$^{-2}$ and $k=200$ lbs. in.$^{-3}$ were fairly representative of the winter condition and that values of $E=4{,}500{,}000$ lbs. in.$^{-2}$ and $k=280$ lbs. in.$^{-3}$ are equally representative of the summer condition of the test sections, values of the coefficients l, D and K were computed for each of the four thicknesses of test section. These computed values are given in table 5. A comparison can be made between these computed values and certain of the values derived from the measured deflections which were shown in table 4. In making such comparisons it is well to bear in mind, first, that the computed values are only as good as the selected values of E and k on which the computations are based, and second, that the computations presuppose the ideal subgrade reaction to obtain. The agreement between the computed values and those derived from the measured deflections is, in general, better for the winter observations than for the summer and better for the interior and edge loadings than for the corner. For the winter condition several comparisons are available and for the interior and edge cases the agreement may be considered good. For the corner case the values of D and K derived from measured deflections tend to be lower than the computed values.

Table 5.—*Computed values of coefficients used in the Westergaard equations*

WINTER CONDITION

[E=5,500,000 lbs. in.$^{-2}$, k=200 lbs. in.$^{-3}$]

Slab thickness	l	D	K
Inches	*Inches*	*Lbs. in.$^{-1}$*	*Lbs. in.$^{-2}$*
6	26.7	142,300	5,340
7	29.9	179,300	5,990
8	33.1	219,100	6,620
9	36.2	261,500	7,230

SUMMER CONDITION

[E=4,500,000 lbs. in.$^{-2}$, k=280 lbs. in.$^{-3}$]

Slab thickness	l	D	K
6	23.3	152,300	6,530
7	26.2	191,900	7,330
8	28.9	234,500	8,100
9	31.6	279,800	8,850

Table 6.—*Displacements caused by rotational movement of the slabs about the longitudinal joint*

Condition of edges of slab	Corner displacement	
	7-in. slab	9-in. slab
	Inches	*Inches*
Warped down	.006	.008
Flat	.015	.016
Warped up	.021	.020

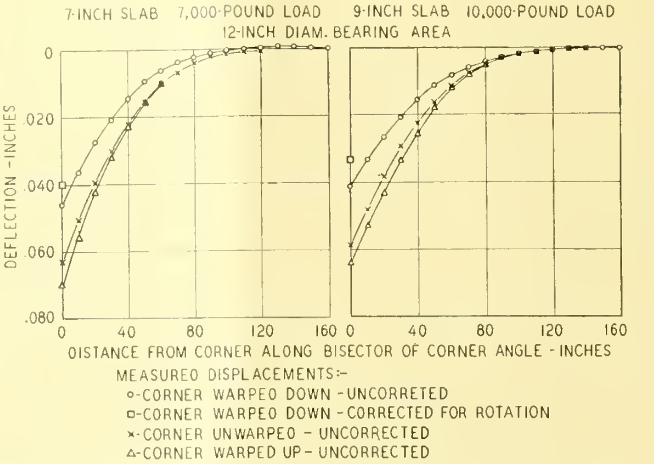


Figure 30.—Deflection of the Bisector of the Corner Angle for Three Conditions of Temperature Warping.

For the summer condition only two comparisons are available, the interior case for the 7-inch and 9-inch sections. In these the values of D and K derived from the measured deflections tend to be 10 to 15 percent lower than the corresponding computed values of table 5. There is generally good agreement in the values of l for the four pavement thicknesses and the three cases of loading.

Further studies of the corner case were made in an effort to establish the reason for the differences between the computed values of the coefficients and those obtained from the measured corner deflections. Tests were made with the 7-inch and 9-inch sections, loads being applied when the corners were flat, warped up and warped down. Measurements were made not only of the deflection but also of the rotational movement of the slab about the longitudinal joint as the corner loads were applied. These rotational movements were measured by placing two recording strain gages on the slab end spanning the joint, one gage near the top and one near the bottom surface of the pavement.

The vertical displacement of the free corner resulting from rotational movement at the longitudinal joint was calculated from the movements recorded by the two strain gages. This displacement is an apparent deflection of the slab corner that must be subtracted from the measured maximum deflection at the corner if the displacement caused by slab flexure alone is to be obtained. Displacement values calculated in this manner for the 7,000-pound load on the corner of the 7-inch test section and the 10,000-pound load on the 9-inch test section are shown in table 6. It will be noted that the values are about the same for the two slab thicknesses but that they vary considerably with the degree of subgrade support afforded the loaded corner, being approximately three times as great for the upward warped corner as for the same corner brought into better contact with the subgrade by downward warping.

The measured total vertical displacements along the bisectors of the corner angle for the 7-inch and 9-inch sections, for the three conditions of temperature warping are shown in figure 30. Each value on the apparent deflection curves in this figure is the total displacement from a fixed datum. The calculated movement of the corner resulting from rotation of the slab about the longitudinal joint is indicated as a correction to be subtracted from the measured total downward movement of the corner point. Because of the rotational movement of the slab about the longitudinal joint it is apparent that the measured load-deflection relation for the corner case cannot be expected to compare directly with the theoretical relation. As nearly as could be determined in this investigation, the measured maximum corner deflection for the downward warped condition when corrected for slab rotation is in reasonably good agreement with the theoretical value. The measured maximum values for the other two conditions of warping are approximately equal after correction and both are approximately 25 percent greater than the corrected value for the downward warped condition.

It is believed that this special study of the deflection of the corners of the 7-inch and 9-inch test sections indicates that, because the measured apparent deflections of the slab corner contain displacements from causes other than flexure, such measurements are not suitable for use in determining the value of the several coefficients previously discussed. One possible exception is the value of the radius of relative stiffness, l, which is determined more by the coincidence of curve shapes than by absolute deflection magnitude. Because of this, values of l determined from corner loadings are in good agreement with values determined from interior and edge loadings for all four slab thicknesses.

LOAD-STRESS RELATIONS ANALYZED

One of the early steps in the testing program was a study of the effect of the size of the bearing area over which the load was distributed on the amount of strain produced by a given load for test sections of various thicknesses and for the three cases of loading considered by Westergaard.

In testing the corners bearing plates having radii, a, of 3, 4, 6, 8, and 10 inches were used. In testing the the interior and edge positions, however, the 6-inch

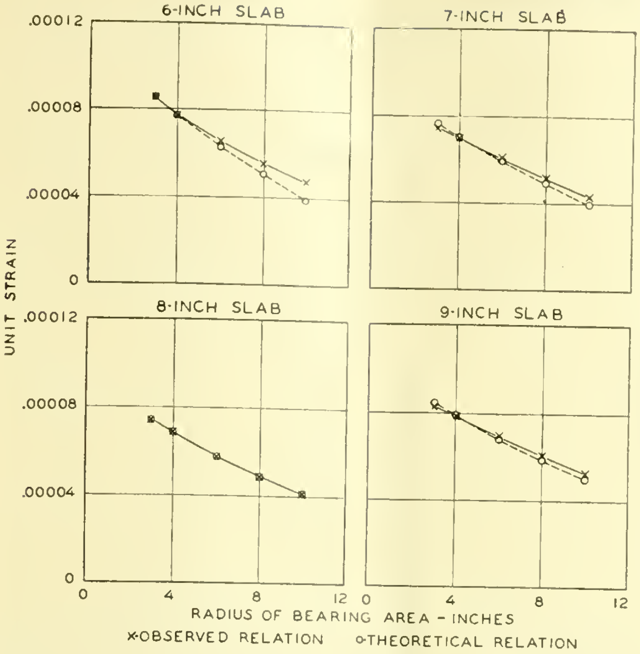


FIGURE 31.—EFFECT OF SIZE OF BEARING AREA ON STRAIN—CORNER LOADING.

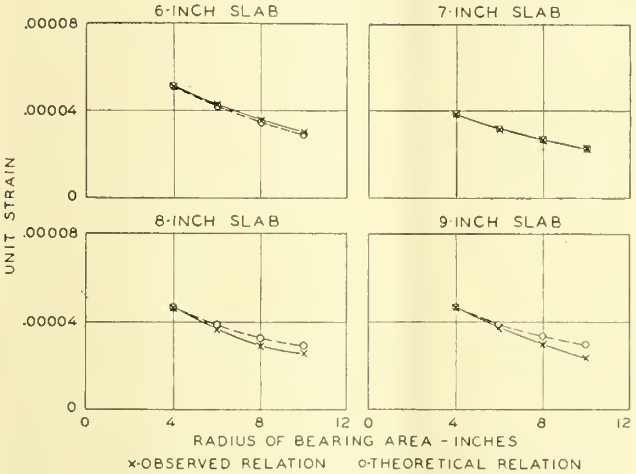


FIGURE 32.—EFFECT OF SIZE OF BEARING AREA ON STRAIN—INTERIOR LOADING.

diameter plate ($a=3$ inches) was omitted because of having to measure strains under the bearing plate with a gage length exceeding the plate diameter. The measured critical strains for each size of bearing area and each slab thickness are shown for the corner, interior and edge loadings in figures 31, 32 and 33, respectively. In order to compare the observed effect of size of bearing area with that indicated by theory, strain values were computed for each value of the radius, a, and these theoretical relations are shown by the broken line curves in the three figures. For purposes of comparison the two curves in each graph were made to coincide for the case of the bearing area with 4-inch radius. For this reason they do not indicate the relative magnitude of observed and computed strains for a given test condition.

In the case of the corner loading (fig. 31) the observed effect of size of bearing area on strain is almost exactly the same as that indicated by theory. The two curves coincide exactly in the comparison for the 8-inch slab while for the other three sections the observed influence of the size of bearing area is only slightly less than the theoretical.

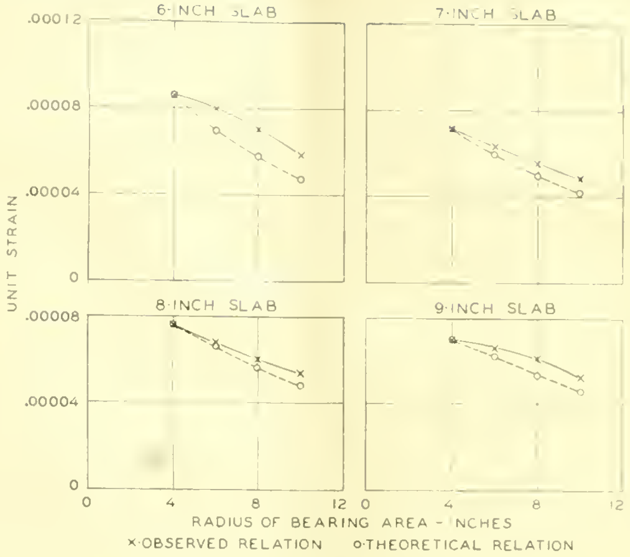


FIGURE 33.—EFFECT OF SIZE OF BEARING AREA ON STRAIN—EDGE LOADING.

For loads applied at the interior (fig. 32) the agreement between the observed and theoretical relations is very close in all cases. Such divergences as appear are so small as to be without significance in interpreting the results.

The comparison between the observed and theoretical relations for the edge loading (fig. 33) is the only one in which a significant difference exists. It will be recalled that in this edge loading (case III) a semicircular bearing area was used in the original analysis. In the test program at Arlington a semicircular bearing plate was used and the effort was made to center the load over the center of area of the plate so as to have uniform load distribution to the slab surface. On the 7-inch and 8-inch test sections the curve showing the observed relation has essentially the same shape as the theoretical curve, although lying somewhat flatter with respect to the x-axis. The curves showing the observed relation for the 6-inch and 9-inch sections on the other hand are of somewhat different shape, being concave with respect to the x-axis instead of convex. The apparent divergence between the observed and theoretical curves for the 6-inch and 9-inch sections is emphasized by the fact that it is the data for the smallest bearing area which appear to be out of line and it so happened that it was at this point that the curves were made to coincide, principally because the 8-inch diameter bearing area was used extensively in the load-strain studies. Even with the discrepancies that have been pointed out for the edge loading there is fair agreement between the experimental data and the theory.

As a general statement it is believed that the observed effect of size of bearing area on strain is so similar to the theoretical relation as to indicate that the latter can be used with confidence in its accuracy.

In subsequent tests where the size of bearing area entered as a variable only the 8-, 12- and 20-inch diameter plates were used.

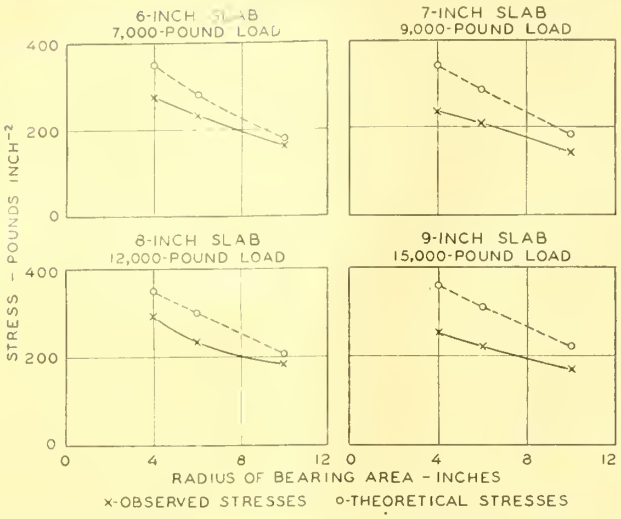


Figure 34.—Comparison of Observed and Theoretical Stresses for Corner Loading Using Coefficients Determined From Deflections Measured at the Corner.

LOAD-STRESS RELATION FOR CORNER LOADING

The first direct comparison between observed and theoretical stresses is shown in figure 34. The term observed stresses used hereafter refers to the critical stress values derived from measured strains, in the manner described earlier in the report. The observed stress values in figure 34 are compared with theoretical values computed with coefficients obtained from corner deflections measured at the same time as the strain measurements. Each observed stress value in this and subsequent figures is based on an average of eight measured strains (2 tests in each quadrant of the test section).

It will be observed in figure 34 that, in every case, the observed stress value is smaller than the theoretical value. This difference is due in large part to the fact that the apparent modulus of elasticity of the concrete, as determined from corner deflection data, is much smaller than the value of the modulus of elasticity determined by other methods. This was brought out in table 4 and the attendant discussion.

When values of the modulus of elasticity obtained from the measured deflections of the interior and edge were averaged and applied to the corner strains for a given test section a better agreement between observed stresses and theoretical stresses resulted. This comparison is shown in figure 35. It is believed that this is a better comparison than that of figure 34 because all evidence indicates that the modulus of elasticity determined from deflection data obtained in tests at the interior and edge is approximately correct. In figure 35 the agreement between the observed and theoretical stress values is very close for the 6-inch and 8-inch sections while for the 7-inch and 9-inch sections the observed stresses are higher than the theoretical stresses by 14 and 18 percent, respectively. The observed influence of the size of bearing area is essentially that indicated by theory.

Some further tests were made to discover, if possible, the reason for the difference between the agreement for the 6- and 8-inch sections and that for the 7- and 9-inch sections as shown in figure 35. The first of these tests were made on the 7- and 9-inch sections during the summer under the condition of maximum downward warping of the slab corners. It is reasonable to assume, therefore, that the pavement corner was in good contact with the subgrade at the time of test. While only one size of bearing plate was used, tests were made on all four quadrants of the test sections and the tests were repeated in two different years. The data may, therefore, be considered representative.

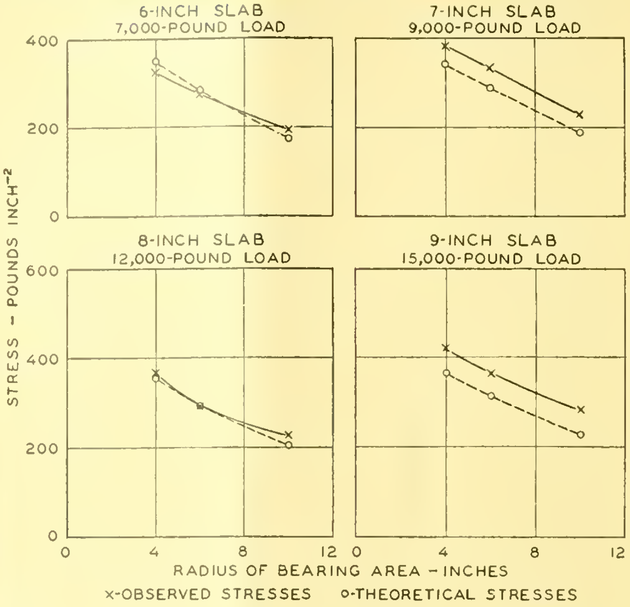


Figure 35.—Comparison of Observed and Theoretical Stresses for Corner Loading Using Average Coefficients Determined From Deflections Measured at the Interior and Edge.

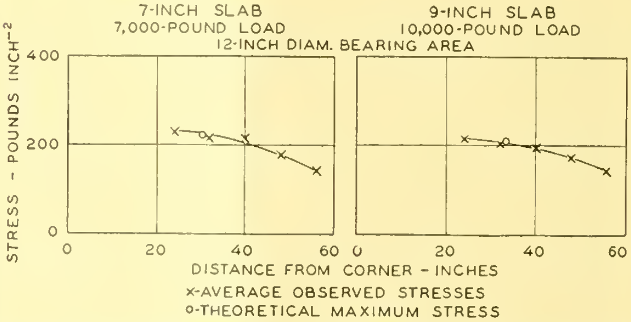


Figure 36.—Comparison of Theoretical Maximum Stress for Corner Loading With Observed Stresses Along the Bisector of the Corner Angle With the Corner Warped Downward.

The average observed stresses along the corner bisector for both the 7-inch and the 9-inch test sections, together with the theoretical maximum stress, are shown in figure 36. Each observed value is an average based on 16 strain measurements. The agreement between the observed and theoretical stress values from these tests indicates that, if the conditions are such that the corner is receiving full subgrade support, values of critical stress computed with the Westergaard equation for critical stress for corner loading (case I) can be used with confidence. When full subgrade support does not exist the computed stresses will be too low.

Further study was given to the effect of the condition of subgrade support on critical stress for corner loading in a series of tests on the 6-inch, 7-inch, and 9-inch sections. In this study loads were applied to the slab corners when those corners were warped upward, unwarped and warped downward. The slabs were as-

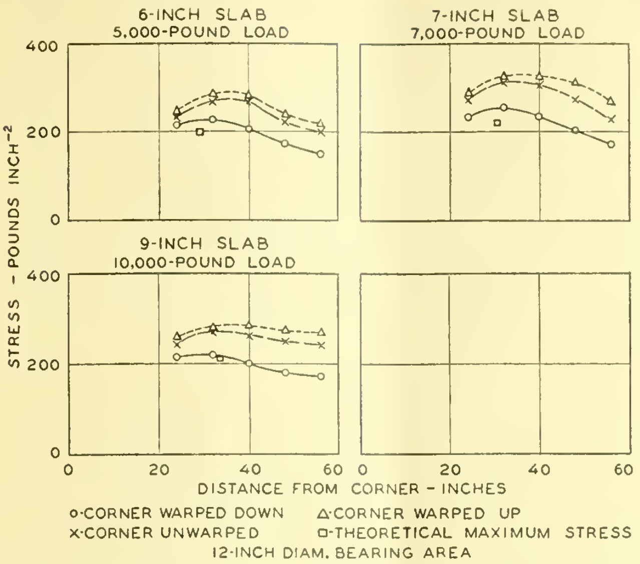


Figure 37.—Average Observed Stresses Along the Bisector of the Corner Angle for Three Conditions of Warping.

sumed to be flat when the temperature of the upper surface and of the lower surface were the same. The 8-inch section was not included because it was necessary to keep the program within practical limits. Conditions for maximum upward warping occur only at certain times in the night or early morning hours in winter and with only three sections included the program extended over nearly one entire winter before the desired data could be obtained. The data from this study are shown in figure 37. Each observed stress value is the result of eight separate strain measurements. The maximum theoretical corner stress is shown on each graph.

Table 7 shows the observed maximum stresses for each condition of warping and for each slab thickness, expressed as a percentage of the theoretical stress. It will be noted that the observed stresses are slightly greater than the theoretical stresses for the condition of downward warping. It is quite possible that even when the corners were warped downward full contact with the subgrade was not established. The observed stresses for the flat and upward warped conditions, i. e., for the conditions of least subgrade support, are higher than the theoretical values by 30 to 50 percent. Thus it is evident that where the assumed condition of full subgrade support does not obtain some correction must be applied to the stress values computed by the theoretical equation for maximum stress from corner loading. Westergaard analyzed the load-deflection and the load-stress relations for slab corners that are not fully supported by introducing a modified condition of subgrade reaction that was assumed to represent the support given a slab corner when that corner was warped upward (*26*).

This analysis provides corrections to be applied to values computed with the equations of the original analysis. In order to compute the corrections it is necessary to determine experimentally certain quantities that appear in the correction equations. A serious effort was made to determine the quantities and the corrections in connection with the investigation being described but for various reasons it was not possible to do so in a satisfactory manner. As a general statement it may be said that the corrections obtained in the manner suggested were too small, at least for the conditions of this investigation.

Table 7.—*Comparison of theoretical corner stresses with maximum observed stresses for three conditions of warping*

Slab thickness	Observed stresses expressed as percentages of theoretical values		
	Corner warped downward	No warping	Corner warped upward
Inches	*Percent*	*Percent*	*Percent*
6	114	137	144
7	116	141	149
9	105	132	138

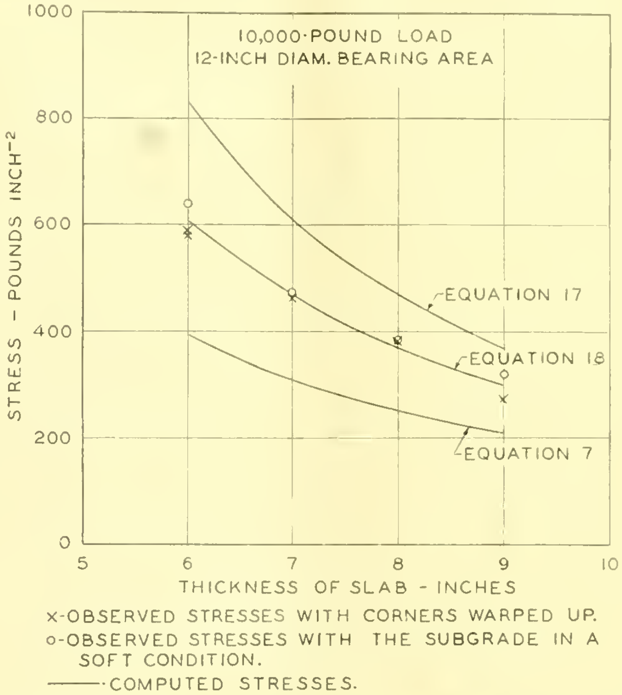


Figure 38.—Comparison of Observed Maximum Corner Stresses With Those Computed by Three Stress Equations for Four Slab Thicknesses.

OBSERVED STRESSES COMPARED WITH VALUES COMPUTED BY VARIOUS STRESS EQUATIONS

In figures 38 and 39 comparisons are made between observed maximum corner stresses and those computed with three different equations which will be described presently. The comparison in figure 38 is for four slab thicknesses, the load and size of loaded area being constant while those for figure 39 are for different sizes of loaded area, with a constant load and slab thickness. As in most of the tests, each observed stress value is the result of eight individual strain measurements. The stress values in figure 38 were either for the condition of upward warping (crosses) or for a very soft subgrade condition (circles) hence are representative of a low corner support condition. Those in figure 39 are for the upward warped condition only. The values of E and k used were selected as representative for the conditions that obtained at the time of the tests.

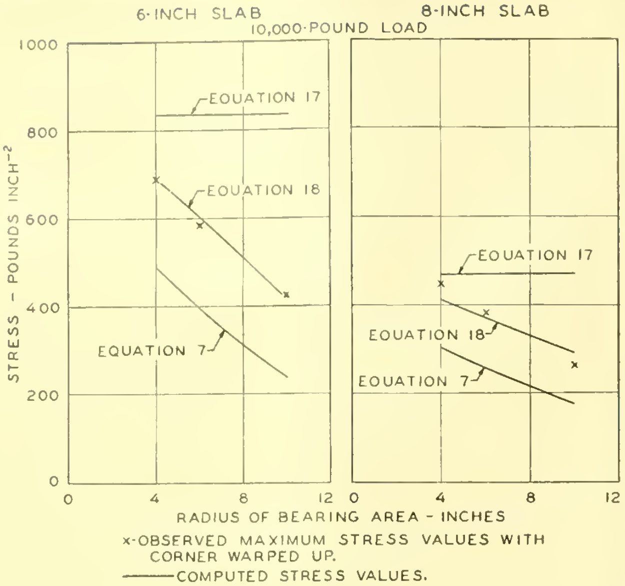


Figure 39.—Comparison of Maximum Observed Corner Stress Values With Those Computed by Three Stress Equations to Show Effect of Size of Bearing Area on Stress.

The three equations referred to above for computing the maximum corner stress values shown in figures 38 and 39 are

$$\sigma_c=\frac{3P}{h^2} \qquad (17)$$

$$\sigma_c=\frac{3P}{h^2}\left[1-\left(\frac{12(1-\mu^2)k}{Eh^3}\right)^{0.15}(a\sqrt{2})^{0.6}\right] \qquad (7)$$

$$\sigma_c=\frac{3P}{h^2}\left[1-\left(\frac{12(1-\mu^2)k}{Eh^3}\right)^{0.3}(a\sqrt{2})^{1.2}\right] \qquad (18)$$

Equation 17 was suggested by Goldbeck (*4*) in 1919 as an approximate formula for computing the stress in concrete slabs. In various forms it came to be known as the "corner formula." It assumes that the slab corner receives no support from the subgrade and that the load is applied at a point at the extreme corner of the slab. In general, stress values computed with this equation are very much higher than those observed in this investigation. In figure 38 the difference will be observed to vary with the pavement thickness, the computed stress being 38, 30, 22, and 24 percent greater respectively than the average observed maximum stress for the corners of the 6-, 7-, 8-, and 9-inch test sections. Since in equation 17 the load is assumed to be concentrated at a point it is obvious that the computed stresses would not be affected by the size of the loaded area. This is apparent in the comparisons of figure 39. For the 6-inch section the computed stresses are greater than those observed for the bearing areas with radii of 4, 6, and 10 inches by 21, 43, and 97 percent respectively. For the 8-inch section the percentages are 4, 23, and 79 respectively for the same three sizes of bearing area.

Equation 7 is the original Westergaard equation for maximum stress for a load applied on the pavement corner over a circular area of radius, *a*, with full subgrade support measured by the reaction modulus *k*. (Note.—It is apparent that equation 17 is a special case of equation 7 for the condition of $k=0$ with *a* infinitely small).

The data presented in figures 38 and 39 support those shown previously in demonstrating that the maximum stresses for corner loading observed in this investigation are considerably greater than the theoretical maxima. If the comparative data from these three figures are averaged it is found that the theoretical values are about one-third less than the observed values and as evidenced by the graphs the difference is fairly constant for various pavement thicknesses and sizes of bearing area.

Equation 18 is an empirical equation developed by modification of the exponents in the Westergaard equation in such a manner as to cause the computed values to coincide more nearly with the observed data for the case of the incompletely supported slab corners shown in figures 38 and 39. The equation thus has no theoretical foundation. It was published earlier in slightly different form (*7*) but the comparisons with observed data were not presented at that time. The data in figure 38 show that for the poorly supported corners to which the observed stress values apply, stress values computed by the empirical equation are in good agreement. The data shown in this figure were obtained in tests using a 12-inch diameter bearing plate. Comparisons with test data obtained with bearing plates of other sizes are shown in figure 39 and the agreement between the observed and computed stress values is generally good.

In addition to the comparisons just shown, it is of interest to note that in a recent report of a laboratory investigation of stress conditions in the corner region of concrete slabs at the Iowa Engineering Experiment Station (*13*) it is stated that the critical corner stresses determined experimentally in that investigation are in general agreement with values calculated by the empirical stress equation developed from the data obtained at Arlington (equation 18).

To summarize, it has been shown that for the corner loading (case I) the conditions of soil support for which the Westergaard equation for maximum stress was developed existed only for short periods of extreme downward warping and that for these periods only was there agreement between the observed stresses and those computed by the Westergaard equation. For the rest of the time the corners were not fully supported and the computed stresses were much lower than the observed stresses.

An empirical equation has been developed that seems to fit the observed data reasonably well and, for conditions of poor corner support, its use is suggested, at least until more comprehensive information is available. It is emphasized, however, that the equation is without theoretical foundation and may not be applicable to all conditions.

LOAD-STRESS RELATION FOR INTERIOR LOADING

A direct comparison between observed and computed stresses for interior loading (case II) is shown in figure 40. The values of observed stress shown in this graph are the critical values for each size of loaded area as determined from strain measurements in the manner previously described and each is the average of eight individual observations. The theoretical stress values in this figure were computed with the equation of the original Westergaard analysis, as restated in generalized

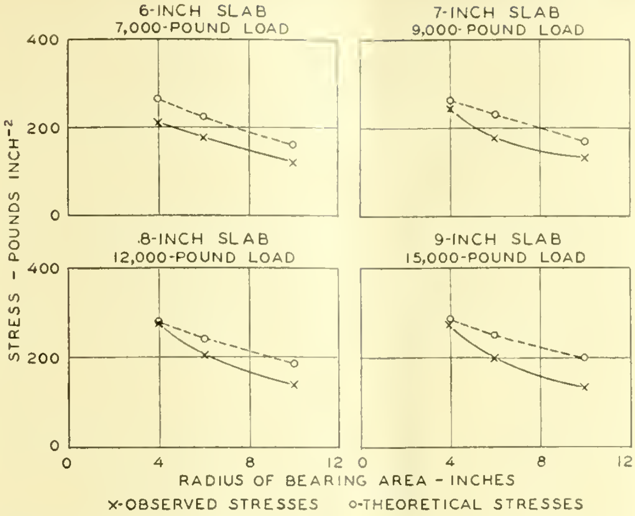


Figure 40.—Comparison of Observed and Theoretical Stresses for Interior Case of Loading $Z=0$.

form (equation 8), the coefficients being determined from deflection measurements of the pavement slab made at the time the strains were measured.

In this comparison the theoretical values are appreciably higher than the observed stresses. The opinion has been expressed by Westergaard that for interior loading the subgrade reactions may be more closely concentrated around the load than are the deflections and that for a given subgrade this redistribution of subgrade reactions results in more support than was contemplated in the original analysis. Based on this conception his "supplementary theory" for interior loading (*25*) was developed. In this supplementary analysis the equation for maximum load stress was modified to the form shown as equation 10 earlier in this discussion. Two new quantities, Z and L, appeared in this equation. Values for these must be determined from experimental data. Westergaard suggests that these constants be established by adjusting their values until fair agreement exists between observed and computed deflection and stresses.

In the present investigation for the subgrade as originally constructed this procedure resulted in the following values: $Z=0.05$ and $L=1.75l$, for the four sections of uniform thickness.

The values of the various coefficients determined from deflections using a value of $Z=0.05$ are shown in table 8. These were developed from the same deflection data as were used for those of table 4 but differ slightly because of the influence of the quantity Z. The values of the modulus of elasticity E, shown in table 8, are in good agreement with those determined by other methods. The values of k, shown in table 8 for the late fall and winter conditions, are in good agreement with the values determined by the bearing tests with rigid plates. For the summer condition the agreement is not as good and a possible explanation for this was offered earlier in the report.

A direct comparison between observed maximum stresses for the interior loading and maximum stress values computed by the Westergaard equation based on a redistribution of subgrade reactions is given in figure 41. Except for the value of L, which as stated above was found to be equal to $1.75l$, the coefficients used in the computation were those given in table 8.

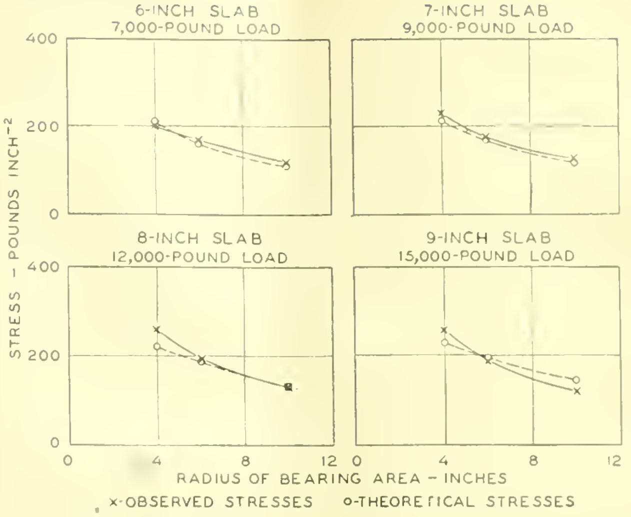


Figure 41.—Comparison of Observed and Theoretical Stresses for Interior Case of Loading $Z=0.05$ and $L=1.75l$.

Table 8.—*Coefficients determined from the deflections for the interior case of loading $Z=0.05$*

Time of testing	Slab thickness	l	k	K	D	E
	Inches	*Inches*	*Lbs. in.*$^{-3}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-1}$	*Lbs. in.*$^{-2}$
Late summer	6	25	185	4, 640	116, 000	3, 930, 000
Winter	7	29	226	6, 550	190, 000	5, 460, 000
Summer	7	28	211	5, 920	165, 700	4, 430, 000
Winter	8	31	247	7, 640	237, 000	5, 220, 000
Late fall	9	36	193	6, 940	250, 000	5, 220, 000
Summer	9	33	210	6, 920	228, 500	4, 000, 000

In general, there is good agreement between the observed and the computed values for all sizes of bearing areas and for all thicknesses of pavement even though only one value of Z and one of L were used in the computation.

So far there has been little or no opportunity to study experimentally the range of variation of these constants to be expected with various concretes and subgrade conditions. Also while the range in size of loaded areas for which data are shown are probably adequate for the usual conditions of highway loading the areas of contact of some airplane tires are quite outside the range. In this connection one of the supplementary studies which is described at the end of this report was made to determine the effect of large bearing areas and of bearing areas of other than circular shape on the load-stress relation and experimental data obtained with larger areas for interior loading are presented in connection with the description of that work.

LOAD-STRESS RELATION FOR EDGE LOADING

A direct comparison between observed stresses and computed stresses for the edge loading is shown in figure 42. The observed stresses are based on measured critical strains while the computed values were obtained with the Westergaard equation for maximum stress for the edge loading, case III, given earlier in this report as equation 9. The coefficients used in the computations were those obtained from deflections made at the time of the strain measurements and are shown in table 4.

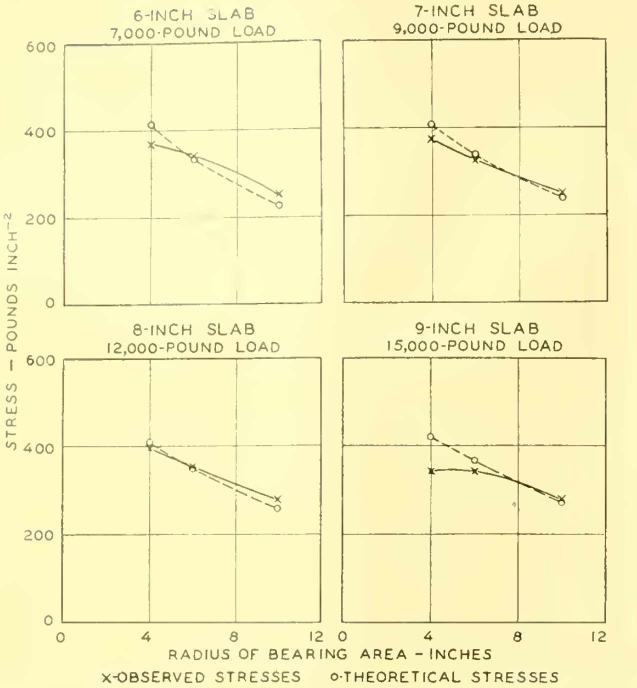


FIGURE 42.—COMPARISON OF OBSERVED AND THEORETICAL STRESSES FOR THE EDGE CASE OF LOADING.

It is apparent from these data that theoretically the effect of size of bearing area is somewhat more pronounced than that observed in these tests although for the 7-inch and 8-inch sections the difference is quite small and the general agreement between the observed and the computed values is quite good. In the case of the 6-inch and 9-inch sections, the agreement is good for the 12-inch and 20-inch diameter bearing areas but for the 8-inch diameter plate the observed stress is definitely lower for both test sections than theory indicates it should be. It will be recalled that this same anomaly was apparent in the strain data shown in figure 33. Just why the stress values for the 8-inch semicircular plate should be low on these sections is not clear. The tests made on the several quadrants of the test sections show no more than the usual spread between individual values so that the average value is not adversely affected by erratic data. Furthermore, repeat tests made 2 years later on the same sections showed essentially the same load-strain values. Even with these values included the general agreement between the observed and computed values may be considered good and it seems reasonable to conclude that the Westergaard equation for maximum stress for edge loading gives an accurate indication when the assumed condition for the loaded area exists and the slab is in an unwarped condition.

Some additional study of the case of an edge load was made to determine the effect of upward warping on the load-stress relation. Some of the tests were made during the late fall with the soil of the subgrade in a moderately wet condition while others were made when the subgrade was soft from thawing after having been frozen to a depth of several inches. The observed stress values from these tests are shown in figures 43 and 44 together with relations computed by two equations for edge stress. The first is the theoretical equation developed by Westergaard for edge loading (case III) the generalized form of which has been given as equation 9. The second equation is as follows

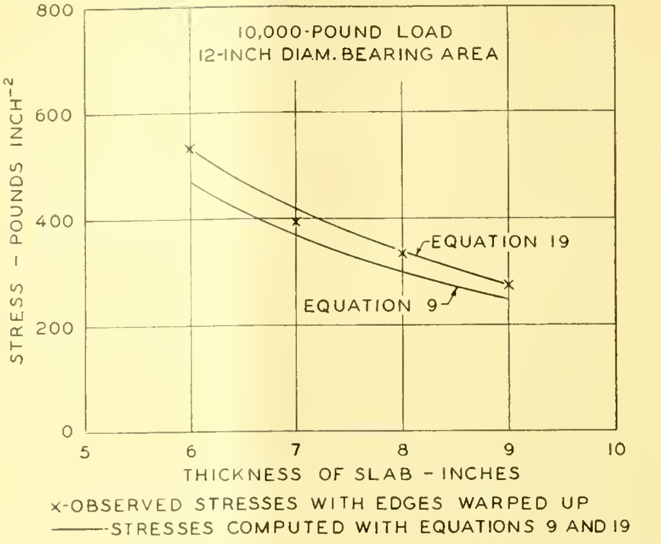


FIGURE 43.—COMPARISON OF OBSERVED MAXIMUM EDGE STRESSES AND EDGE STRESSES COMPUTED BY THEORETICAL EQUATION 9 AND EMPIRICAL EQUATION 19 FOR FOUR SLAB THICKNESSES.

$$\sigma_e=0.529(1+0.54\mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{Eh^3}{kb^4}\right)+\log_{10}\left(\frac{b}{1-\mu^2}\right)-1.0792\right]\text{------------}(19)$$

and is an empirical equation developed to fit the observed stress values obtained in load tests on the upward warped slab edges for conditions of subgrade support such as prevailed when these particular tests were made. The same empirical equation, in slightly different form, has been published previously (7) but the data from which it was developed were not presented at that time.

Figure 43 shows the effect of slab thickness on the maximum stress from edge loading while in figure 44 the effect on stress magnitude of the size of the loaded area is brought out. As in the graphs shown previously each observed stress value is an average derived from eight individual strain measurements, two on each of the four quadrants of the test section. From the data in this figure it is apparent that the observed stresses, for the conditions of the test, exceed the values computed by the Westergaard equation for maximum edge stress by about 10 percent. In making this comparison it is well to keep in mind that upward warping of slab edges develops only when the bottom surface of the slab is warmer than the upper surface. Appreciable upward warping occurs relatively infrequently in the region where these tests were made. The tests for which the data are shown in figure 43 were made before sunrise on winter mornings when the cycle of temperature changes happened to be favorable. At the time of year during which the observed data were obtained the supporting value of the subgrade is apt to be lessened by a high moisture content. In view of the critical conditions represented it is rather surprising that the observed stress values do not exceed those computed by equation 9 by a greater percentage.

The observed stress values are in good agreement with the values computed by the empirical equation for maximum edge stress, equation 19, for all four pavement thicknesses represented in the investigation.

In figure 44 the observed data indicate that, when the slab edge is warped up the effect of the size of loaded area, as measured by the radius, *a*, is slightly less pronounced than theory indicates. The values are higher than those computed by the theoretical equation (equation 9) also, as would be expected in the light of the preceding discussion. Stress values computed by the empirical equation (equation 19) agree closely with the observed stresses for the 12-inch and 20-inch diameter bearing areas but are somewhat higher than the stresses observed in the tests with the 8-inch diameter area, the average difference being of the order of 10 percent.

The empirical equation was derived by adjusting the theoretical relation expressed by the Westergaard equation for maximum edge stress until the empirical relation fitted the observed data as closely as possible. The empirical equation is, therefore, specific rather than general in its application. However, it yields stress values that are somewhat higher, for given conditions, than those computed with the theoretical equation, a difference that would be expected to exist generally when the edge of the slab is warped upward. Therefore, if this extreme condition is being considered, the use of the empirical equation is suggested, at least until such time as more comprehensive data are available.

RESULTS OF INVESTIGATION REVIEWED

Before presenting the discussion of certain supplementary studies of pavement stresses that were made as a part of this investigation it is desirable to review briefly and to discuss the general aspects of the material which has been presented up to this point.

The Westergaard analyses of the load-deflection and load-stress relations in concrete pavement slabs of uniform thickness, resting on an elastic subgrade, have been studied experimentally. Comparisons have been made between computed and observed deflections and between computed and observed stresses on full-size pavement sections of 6-, 7-, 8- and 9-inch uniform thicknesses. Studies have been made of methods for determining the various coefficients and other quantities that appear in the Westergaard equations.

As a result of this study the general conclusion is drawn that the Westergaard analysis expresses quite accurately the relations between load and deflection and between load and critical stress for various thicknesses of pavement and for various sizes of bearing area provided the slab is in full contact with the subgrade. The subgrade stiffness coefficient and other quantities must of course be determined for the particular conditions that exist.

It has been found possible to determine usable values of the modulus of subgrade reaction, *k*, by means of load-displacement tests with rigid bearing plates. In making such tests it is necessary to use plates of adequate size and to limit the displacement to a value that approximates the pavement deflection under load.

The experimental determination of the other quantities that appear in the equations has been found to be feasible although the facility with which the determinations may be made and the precision of the values obtained is not the same for all quantities. An uncertain factor is the effect of moisture variation in the concrete of the pavement. Moisture variation affects the stiffness of the concrete and if a moisture differential exists it may affect the state of stress through restrained warping. The influence of variation in concrete stiffness is relatively unimportant so far as the use of the theoretical equations is concerned but has a direct effect when measured strains are being converted into stresses. No satisfactory means has yet been found for indicating the moisture conditions that exist in a concrete pavement slab in place on the subgrade.

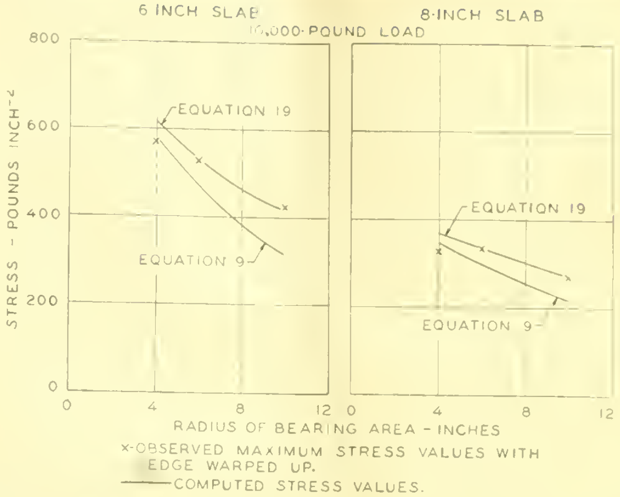


FIGURE 44.—COMPARISON OF OBSERVED MAXIMUM EDGE STRESSES AND EDGE STRESSES COMPUTED BY THEORETICAL EQUATION 9 AND EMPIRICAL EQUATION 19 FOR THREE SIZES OF BEARING AREA.

Unless the values for the quantities that appear in the stress equations have been determined with great care too great reliance should not be placed on the absolute value of computed deflections or stresses.

For conditions where the slab corners or edges are not in full contact with the subgrade because of temperature warping or other cause, empirical equations have been developed that fit the data obtained in tests reported herein. These equations are not offered as a replacement for the theoretical equations of Westergaard but it is thought they may prove useful in certain studies where incomplete subgrade support exists or is assumed.

USE OF THE EQUATIONS

In the second report of this series (*19*) it was shown that the stresses caused by restrained temperature warping can be large and that the magnitude of these stresses depends not only on the temperature differential between the upper and lower surfaces of the pavement but also on the length and thickness of the pavement unit or slab. The greatest temperature differentials develop during the afternoon of clear days in late spring and early summer, the temperature of the upper surface being as much as 20° to 30° Fahrenheit or more above that of the lower surface at such times (for the vicinity of Washington, D. C.). This condition causes the edges of the slab to tend to warp downward and, as this tendency is restrained, a tensile stress is developed in the bottom of the slab. When the upper surface of the pavement is at a temperature lower than that of the bottom the tendency for an upward warping of the slab edges is created. This condition may develop during the night and early morning hours from time to time throughout the year. The magnitude of the maximum temperature differential developed at night usually does not exceed about one-third of that which may develop during the day. The restraint to the upward warping of the slab edges caused by the weight of the slab or by the joint construction produces tensile stress in the upper surface of the slab.

These stresses which develop as a result of restraint to warping combine with the maximum stresses developed by wheel loads. An adequate slab design, therefore, is one which will satisfactorily withstand the most critical combination of load and warping stresses. It has been shown (*19*) that the most effective means for controlling the magnitude of the temperature warping stresses in concrete pavements is by limiting the length and width of the panels or slab units and maximum values of the order of 10 to 15 feet were suggested. Even with slab units limited to dimensions of 10 or 15 feet, however, the stresses from restrained temperature warping are not eliminated. Their magnitude is simply controlled to a reasonable maximum.

This brief discussion of temperature warping has been included because it is believed that it is a subject which must be considered in most computations of pavement slab stresses. For a more thorough discussion the reader is referred to published reports (*24, 19, 7*).

For computing the maximum stress for the corner loading Westergaard developed the generalized theoretical equation given earlier in the report and for convenience repeated here.

Theoretical Equations (*Case I*)

$$\sigma_c=\frac{3P}{h^2}\left[1-\left(\frac{12(1-\mu^2)k}{Eh^3}\right)^{0.15}(a\sqrt{2})^{0.6}\right]\quad(7)$$

This can be restated in terms of l as follows:

$$\sigma_c=\frac{3P}{h^2}\left[1-\left(\frac{a\sqrt{2}}{l}\right)^{0.6}\right]$$

These equations give the most accurate indication of maximum load stress when the pavement corner is in full contact with the subgrade. In this investigation the condition was attained only when the corner was warped downward. If these equations are used for computing load stress the condition of corner warping due to temperature would be such as to create a moderate compressive stress in the upper surface of the slab in the region where the load would create the maximum tensile stress. Thus the combined stress would tend to be slightly lower than the load stress alone.

When the slab corner is not in complete bearing on the subgrade, due to upward warping, the theoretical equation will give load-stress values somewhat lower than those that will be developed. For this condition the Arlington experiments indicate the empirical equation, repeated below, will give computed values that are more nearly in accord with those observed.

Empirical Equations (*Case I*)

$$\sigma_c=\frac{3P}{h^2}\left[1-\left(\frac{12(1-\mu^2)k}{Eh^3}\right)^{0.3}(a\sqrt{2})^{1.2}\right]\quad(18)$$

or expressed in terms of l

$$\sigma_c=\frac{3P}{h^2}\left[1-\left(\frac{a\sqrt{2}}{l}\right)^{1.2}\right]$$

As a general rule the most critical condition for the corner loading is at night when the corner tends to warp upward. The subgrade support is least effective at that time and any warping stress present in the corner is additive to the load stress.

The case of interior loading is covered by the original Westergaard stress equation in generalized form, given as equation 8 earlier in the paper, or by the equation which was developed for the modified condition of subgrade support in the supplementary paper (*25*) and given in this report as equation 10.

Theoretical Equations (*Case II*)

$$\sigma_i''=0.275(1+\mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{Eh^3}{kb^4}\right)-54.54\left(\frac{l}{L}\right)^2 Z\right]\quad(10)$$

If $\mu=0.15$ and $Z=0$ this equation can be simply expressed in terms of l, as follows:

$$\sigma_i=0.316\frac{P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right)+1.069\right]$$

The load stresses at the interior of a slab are not appreciably affected by the condition of warping of the slab and the same equation may be used for either upward or downward warping. Temperature warping stresses are highest in the interior region of the slab, however, and the combined stress value may vary widely from night to day.

There is little information at the present time to indicate what values of Z and L should be used in the general equation for various conditions of pavement and subgrade stiffness. It will be recalled that the data presented in figure 41 showed good agreement between observed and computed stress values when $Z=0.05$ and $L=1.75l$. These values were determined from the deflections and stresses observed on all of the four different thicknesses of pavement and there was no apparent difference in the value of either coefficient for the various thicknesses of slab. As stated previously, however, there is no information to indicate what values of the coefficients should be applied to other concretes and other subgrades. When dealing with conditions that are not known the values assigned to the coefficients Z and L should be such as to result in conservative load stresses. For such use the values $Z=0.2$ and $L=5l$ have been suggested (*7*) pending the development of more comprehensive data.

The stresses resulting from edge loading, like those from corner loading are affected by the degree of warping present at the time the load is applied. The theoretical equation for edge stress as given in generalized form by Westergaard appeared as equation 9 earlier in the report, as follows:

Theoretical Equations (*Case III*)

$$\sigma_e=0.529(1+0.54\mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{Eh^3}{kb^4}\right)-0.71\right]\quad(9)$$

If $\mu=0.15$ it can be expressed simply in terms of l:

$$\sigma_e=0.572\frac{P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right)+0.360\right]$$

In the Arlington tests it was found that under critical conditions of subgrade support and upward warping of the slab edges, the values computed by this equation were somewhat low and an empirical equation was developed which fitted the observed stress values more closely.

Empirical Equations (Case III)

$$\sigma_e=0.529(1+0.54\mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{Eh^3}{kb^4}\right)+\log_{10}\left(\frac{b}{1-\mu^2}\right)-1.079\right] \quad (19)$$

This equation also can be expressed in terms of l, when $\mu=0.15$, as follows:

$$\sigma_e=0.572\frac{P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right)+\log_{10}b\right]$$

The critical stress from edge loading is a tension in the bottom edge of the pavement directly under the loaded area in a direction parallel to the pavement edge. Temperature warping stress in this direction can at times be high in the region of the slab edge and rather wide variations in combined stress from night to day may occur. The most critical condition for edge loading is that which occurs when the stress created during downward warping of the pavement edge (during the day) is combined with load stress. For this case the load stress would be computed with the theoretical equation (equation 9).

TABLE 9.—*Stresses computed by the Westergaard equation for the case of a corner loading and full subgrade support*

$$\sigma_c=\frac{3P}{h^2}\left[1-\left(\frac{12(1-\mu^2)k}{Eh^3}\right)^{0.15}(a\sqrt{2})^{0.6}\right] \quad (7)$$

[P=10,000 pounds. μ=0.15]

Thickness of slab, h	Modulus of subgrade reaction, k	Maximum load stress																			
		E=3,000,000 a in inches—					E=4,000,000 a in inches—					E=5,000,000 a in inches—					E=6,000,000 a in inches—				
		2	4	6	8	10	2	4	6	8	10	2	4	6	8	10	2	4	6	8	10
Inches	*Lbs. in.*$^{-3}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$
6	50	641	541	461	390	327	649	553	476	409	348	655	563	488	423	364	660	570	497	434	377
	100	619	509	420	342	271	628	523	437	362	295	635	533	450	378	313	641	540	461	390	327
	200	596	474	375	288	210	606	489	394	311	236	613	500	408	328	257	619	509	420	342	271
	300	581	451	346	254	171	592	467	366	278	199	600	479	382	296	219	606	489	394	311	236
7	50	480	412	357	309	265	486	420	367	321	280	490	427	376	331	291	493	432	382	339	299
	100	466	390	329	275	227	472	399	341	289	243	476	406	350	300	255	479	412	357	309	265
	200	450	366	298	238	185	456	376	311	254	203	462	384	321	266	216	465	390	329	275	228
	300	439	350	278	215	158	447	361	292	232	177	452	369	303	244	191	453	376	311	254	204
8	50	373	325	285	250	218	377	330	292	259	229	380	335	298	266	237	383	339	303	271	243
	100	363	309	265	226	191	367	315	273	236	203	371	320	279	244	211	373	324	285	250	218
	200	352	291	242	199	161	356	298	251	211	174	360	304	259	219	183	363	309	264	226	191
	300	344	280	228	182	141	349	288	238	194	155	353	294	245	203	165	356	298	252	211	174
9	50	299	262	233	206	183	302	267	238	213	191	304	270	242	218	197	305	273	246	223	201
	100	291	250	217	188	162	294	255	224	196	171	297	259	228	202	178	298	262	232	206	183
	200	282	237	201	168	139	286	243	208	177	149	289	247	213	183	156	290	250	217	188	162
	300	277	229	190	156	125	281	235	197	165	135	284	239	203	172	143	285	243	208	177	149

TABLE 10.—*Stresses computed by a modified equation for the case of corner loading and deficient subgrade support*

[NOTE: This is a purely empirical modification of the Westergaard corner equation, developed on the basis of experimental stress determinations, for the case of a slab corner which does not receive full subgrade support.]

$$\sigma_c=\frac{3P}{h^2}\left[1-\left(\frac{12(1-\mu^2)k}{Eh^3}\right)^{0.3}(a\sqrt{2})^{1.2}\right] \quad (18)$$

[P=10,000 pounds. μ=0.15]

Thickness of slab, h	Modulus of subgrade reaction, k	Maximum load stress																			
		E=3,000,000 a in inches—					E=4,000,000 a in inches—					E=5,000,000 a in inches—					E=6,000,000 a in inches—				
		2	4	6	8	10	2	4	6	8	10	2	4	6	8	10	2	4	6	8	10
Inches	*Lbs. in.*$^{-3}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$
6	50	789	731	667	598	525	792	739	680	617	551	795	745	690	631	569	797	750	698	642	583
	100	778	707	628	543	454	783	718	645	567	486	786	725	657	585	508	789	731	667	598	525
	200	766	678	581	476	367	771	691	601	506	405	775	700	616	527	433	778	707	628	543	454
	300	757	658	548	430	306	763	672	571	463	350	768	683	588	487	381	771	691	601	506	405
7	50	584	547	506	462	415	586	552	514	474	432	588	556	521	483	443	589	559	526	490	452
	100	577	532	481	427	370	580	538	492	442	390	582	543	500	453	404	584	547	506	462	415
	200	569	513	451	384	314	573	521	464	403	339	575	527	474	416	356	577	532	481	427	370
	300	563	500	430	354	275	567	509	445	376	303	570	516	456	391	323	573	521	464	403	338
8	50	449	424	396	367	335	451	428	402	375	346	452	431	407	381	354	453	433	410	386	360
	100	445	414	380	343	304	447	419	387	353	318	448	422	392	361	328	449	424	396	367	[illegible]
	200	439	401	359	314	266	442	407	368	327	283	444	411	375	336	295	445	414	380	343	304
	300	436	393	345	294	240	438	399	355	308	259	440	403	362	319	272	442	407	368	327	283
9	50	357	339	319	298	275	358	341	323	304	283	359	343	326	308	289	359	345	329	311	293
	100	353	331	307	281	253	355	335	312	288	263	356	337	316	294	270	357	339	319	298	275
	200	350	322	292	260	226	351	326	299	269	238	353	329	303	276	247	353	[illegible]	307	281	253
	300	347	316	282	246	208	349	321	290	256	221	350	324	295	264	231	351	[illegible]	299	269	238

TABLES OF COMPUTED STRESSES PREPARED TO SHOW EFFECT OF VARIABLES IN EQUATIONS

In order to show the manner in which the various quantities that appear in the equations for maximum stress affect the computed values tables 9 to 16 have been prepared. Tables 9 and 10 apply to corner loading, case I, and show stress values for full and partial subgrade support respectively. Tables 11 and 12 show stress values for the interior loading, case II, for two assumptions regarding the distribution of the subgrade reactions within the deflected area. The stress values of table 11 are based on the original Westergaard assumptions (Z being 0) while those of table 12 were computed for values of Z and L determined experimentally in this investigation. Tables 13 and 14 give computed stress values for edge loading, case III, for a fully supported edge and for one that has warped upward and has only partial subgrade support. The equations used for computing the stress values in tables 10 and 14 are empirical, developed to fit the experimental data of this investigation. The stress values of table 12 although computed with a theoretical equation are based on constants determined experimentally for the conditions that existed at Arlington. The stress values of these three tables (10, 12, and 14) may be limited in their application, therefore.

Tables 15 and 16, computed by equations 8 and 9 respectively, show the effect of varying the value of several of the coefficients on the computed stress magnitude for interior loading, case II, and edge loading, case III.

TABLE 11.—*Stresses computed by the Westergaard equation for the case of interior loading for the case $Z=0$*

$$\sigma_i=0.275(1+\mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{Eh^3}{kb^4}\right)\right] \qquad (8)$$

[P=10,000 pounds. μ=0.15]

Thickness of slab, h	Modulus of subgrade reaction, k	Maximum load stress																			
		E=3,000,000 a in inches —					E=4,000,000 a in inches —					E=5,000,000 a in inches —					E=6,000,000 a in inches —				
		2	4	6	8	10	2	4	6	8	10	2	4	6	8	10	2	4	6	8	10
Inches	*Lbs. in.*$^{-3}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$
6	50	487	421	361	313	274	498	432	372	324	285	507	441	381	333	294	514	448	388	340	301
	100	461	395	335	287	248	472	406	346	298	259	480	414	354	306	267	487	421	361	313	274
	200	435	368	308	260	221	446	379	319	271	232	454	388	328	280	241	461	395	335	287	248
	300	419	353	293	245	206	430	364	304	256	217	439	372	312	264	225	446	379	319	271	232
7	50	360	319	279	245	217	368	327	287	253	225	374	333	293	259	231	379	339	298	264	236
	100	341	300	260	226	197	349	308	268	234	206	355	314	274	240	212	360	319	279	245	217
	200	321	280	240	206	178	329	288	248	214	186	335	295	254	220	192	341	300	260	226	198
	300	310	269	229	195	167	318	277	237	203	175	324	283	243	209	181	329	288	248	214	186
8	50	276	250	222	197	176	282	256	228	203	182	287	261	233	208	187	291	265	237	212	191
	100	261	235	207	182	161	267	241	213	188	168	272	246	218	193	172	276	250	222	197	176
	200	246	220	192	167	146	252	226	198	174	153	257	231	203	178	157	261	235	207	182	161
	300	238	211	183	159	138	244	217	189	165	144	249	222	194	170	149	252	226	198	174	153
9	50	218	200	180	162	146	223	205	185	167	151	227	209	189	171	155	230	212	192	174	158
	100	206	188	169	150	134	211	193	173	155	139	215	197	177	159	143	218	200	180	162	146
	200	194	177	157	138	123	199	182	162	143	127	203	185	166	147	131	206	188	169	150	134
	300	188	170	150	132	116	192	175	155	137	121	196	179	159	140	124	199	182	162	143	127

TABLE 12.—*Stresses computed by the Westergaard equation for the case of interior loading*

$$\sigma_i''=0.275(1+\mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{Eh^3}{kb^4}\right)-54.54\left(\frac{l}{L}\right)^2Z\right] \qquad (10)$$

[P=10,000 pounds. μ=0.15. Z=0.05. L=1.75l]

Thickness of slab, h	Modulus of subgrade reaction, k	Maximum load stress																			
		E=3,000,000 a in inches—					E=4,000,000 a in inches—					E=5,000,000 a in inches—					E=6,000,000 a in inches—				
		2	4	6	8	10	2	4	6	8	10	2	4	6	8	10	2	4	6	8	10
Inches	*Lbs. in.*$^{-3}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$
6	50	409	343	283	235	196	420	354	294	246	207	429	362	303	254	216	436	369	310	261	222
	100	383	317	257	209	170	394	327	268	219	181	402	336	276	228	189	409	343	283	235	196
	200	357	290	230	182	143	367	301	241	193	154	376	310	250	201	163	383	317	257	208	170
	300	341	275	215	166	128	352	286	226	177	139	360	294	234	186	147	367	301	241	193	154
7	50	303	262	222	188	160	311	270	229	196	168	317	276	236	202	174	322	281	241	207	179
	100	284	243	203	169	140	291	250	210	176	148	297	257	216	182	154	303	262	221	188	159
	200	264	223	183	149	121	272	231	191	157	129	278	237	197	163	135	283	242	202	168	140
	300	252	211	171	137	109	260	220	179	145	117	267	226	186	152	124	272	231	191	157	129
8	50	232	206	178	153	132	238	212	184	159	138	243	217	189	164	143	247	221	193	168	147
	100	217	191	163	138	117	223	197	169	144	124	228	202	174	149	128	232	206	178	153	132
	200	202	176	148	123	102	208	182	154	130	109	213	187	159	134	113	217	191	163	138	117
	300	194	167	139	115	94	200	173	145	121	100	205	178	150	126	105	208	182	154	130	109
9	50	183	165	145	127	111	188	170	150	132	116	192	174	154	136	120	195	177	157	139	123
	100	171	153	134	115	99	176	159	139	120	104	180	162	142	124	108	183	165	146	127	111
	200	159	142	122	103	88	165	147	127	109	93	168	151	131	112	96	171	154	134	115	99
	300	153	135	115	97	81	158	140	120	102	86	161	144	124	105	90	165	147	127	109	93

TABLE 13.—*Stresses computed by the Westergaard equation for the case of edge loading and full subgrade support*

$$\sigma_e=0.529(1+0.54\mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{Eh^3}{kb^4}\right)-0.71\right] \qquad (9)$$

[P=10,000 pounds. μ=0.15]

Thickness of slab, *h*	Modulus of subgrade reaction, *k*	Maximum load stress																			
		E=3,000,000 *a* in inches—					*E*=4,000,000 *a* in inches—					*E*=5,000,000 *a* in inches—					*E*=6,000,000 *a* in inches				
		2	4	6	8	10	2	4	6	8	10	2	4	6	8	10	2	4	6	8	10
Inches	*Lbs. in.*$^{-3}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$
6	50	769	649	541	453	383	789	669	561	473	403	804	684	576	489	419	817	697	589	501	431
	100	721	601	493	406	335	741	621	513	426	355	756	637	528	441	371	769	649	541	454	383
	200	673	553	445	358	287	693	573	465	378	307	708	589	480	393	323	721	601	493	406	336
	300	645	525	417	330	260	665	545	437	350	279	680	561	452	365	295	693	573	465	378	307
7	50	568	494	422	360	309	583	509	436	375	324	594	520	448	386	335	603	530	457	395	345
	100	533	459	386	325	274	548	474	401	340	289	559	485	412	351	300	568	494	422	360	310
	200	498	424	351	290	239	513	439	366	305	254	524	450	377	316	265	533	459	386	325	274
	300	477	404	331	269	219	492	418	345	284	233	503	429	357	295	245	513	439	366	305	254
8	50	436	388	337	293	255	447	399	349	304	266	456	408	357	313	275	463	415	365	320	282
	100	409	361	311	266	228	420	372	322	277	239	429	381	331	286	248	436	388	338	293	255
	200	382	334	284	239	201	393	345	295	250	213	402	354	304	259	221	409	361	311	266	228
	300	366	319	268	223	186	377	330	279	235	197	386	338	288	243	205	393	345	295	250	213
9	50	344	312	276	243	214	353	321	285	252	223	360	328	292	259	230	365	333	297	264	235
	100	323	291	255	222	193	332	300	264	230	202	338	306	270	237	208	344	312	276	243	214
	200	301	269	233	200	171	310	278	242	209	180	317	285	249	216	187	323	291	255	222	193
	300	289	257	221	188	159	298	266	230	197	168	305	273	237	204	175	310	278	242	209	180

TABLE 14.—*Stresses computed by a modified equation for the ease of edge loading and deficient subgrade support*

[NOTE: This is a purely empirical modification of the Westergaard edge equation, developed on the basis of experimental stress determinations, for the case of a slab edge which does not receive full subgrade support.]

$$\sigma_e=0.529(1+0.54\mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{Eh^3}{kb^4}\right)+\log_{10}\left(\frac{b}{1-\mu^2}\right)-1.0792\right] \qquad (19)$$

[P=10,000 pounds. μ=0.15]

Thickness of slab, *h*	Modulus of subgrade reaction, *k*	Maximum load stress																			
		E=3,000,000 *a* in inches—					*E*=4,000,000 *a* in inches—					*E*=5,000,000 *a* in inches—					*E*=6,000,000 *a* in inches—				
		2	4	6	8	10	2	4	6	8	10	2	4	6	8	10	2	4	6	8	10
Inches	*Lbs. in.*$^{-3}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$
6	50	774	684	603	537	485	793	704	622	557	504	809	719	638	572	520	821	732	650	585	532
	100	726	636	555	489	437	746	656	575	509	457	761	671	590	525	472	774	684	603	537	485
	200	678	588	507	442	389	698	608	527	461	409	713	623	542	477	424	726	636	555	489	437
	300	650	560	479	414	361	670	580	499	433	381	685	595	514	449	396	698	608	527	461	409
7	50	577	521	467	421	383	591	536	481	435	397	603	547	493	447	409	612	556	502	456	418
	100	542	486	432	386	348	556	501	446	400	362	567	512	458	412	374	577	521	467	421	383
	200	506	451	397	351	313	521	466	411	365	327	532	477	422	376	338	542	486	432	386	348
	300	486	431	376	330	292	500	445	391	345	307	512	456	402	356	318	521	466	411	365	327
8	50	446	410	372	339	311	457	421	384	350	322	466	430	392	359	330	473	437	399	366	338
	100	419	383	346	312	284	430	395	357	323	295	439	403	365	332	301	446	410	372	339	311
	200	392	356	319	285	257	403	368	330	296	268	412	376	338	305	277	419	383	346	312	284
	300	376	341	303	269	241	388	352	314	281	252	396	361	323	289	261	403	368	330	296	268
9	50	355	331	304	279	257	364	340	313	288	266	371	347	320	295	273	376	352	325	300	279
	100	334	310	283	258	236	342	318	292	267	245	349	325	298	273	252	355	331	304	279	257
	200	312	288	261	237	215	321	297	270	245	224	328	304	277	252	231	334	310	283	258	236
	300	300	276	249	224	202	309	285	258	233	211	316	292	265	240	218	321	297	270	245	224

As will be noted the computed stress values in table 11 are for a value of $Z=0$ (and $L=0$). The values may be corrected for other values of Z and L, rather simply, by adding a stress correction, σ_i', computed with the following equation:

$$\sigma_i'=-\frac{15(1+\mu)ZP}{h^2}\left(\frac{l}{L}\right)^2$$

The stress values in tables 9 to 14, inclusive, were computed for a value of $\mu=0.15$. The maximum stress for corner loading is affected very little by changes in μ within the normal range of variation of this ratio. The effect of changes in the value of μ for the interior and edge cases of loading is shown in tables 15 and 16 respectively. The effect is not large and if it is desired to obtain computed stress values for intermediate values of the ratio, direct interpolation is sufficiently accurate for all practical purposes.

TABLE 15.—*Stresses computed for the case of interior loading to show the effects of variations in the values of h, k, a, E and μ*

[P=10,000 pounds. Z=0]

Thickness of slab, *h*	Modulus of subgrade reaction, *k*	Maximum load stress								
		E=3,000,000 *a* in inches—			*E*=5,000,000 *a* in inches—			*E*=6,000,000 *a* in inches—		
		2	6	10	2	6	10	2	6	10
Inches	*Lbs. in.*$^{-3}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$
		μ=0.05								
6	100	421	306	226	439	324	244	445	330	250
	200	397	282	202	415	299	220	421	306	226
	300	383	268	188	400	285	206	407	292	212
8	100	238	189	147	248	199	157	252	202	161
	200	225	175	134	235	185	144	238	189	147
	300	217	167	126	227	177	136	230	181	139
9	100	188	154	123	196	162	131	199	165	133
	200	178	143	112	185	151	120	188	154	123
	300	171	137	106	179	145	114	182	148	116
		μ=0.15								
6	100	461	335	248	480	354	267	487	361	274
	200	435	308	221	454	328	241	461	335	248
	300	419	293	206	439	312	225	446	319	232
8	100	261	207	161	272	218	172	276	222	176
	200	246	192	146	257	203	157	261	207	161
	300	238	183	138	249	194	149	252	198	153
9	100	206	169	134	215	177	143	218	180	146
	200	194	157	123	203	166	131	206	169	134
	300	188	150	116	196	159	124	199	162	127
		μ=0.25								
6	100	501	364	269	522	385	291	530	393	298
	200	472	335	241	493	356	262	501	364	269
	300	455	318	224	477	340	245	484	347	253
8	100	284	225	175	296	237	187	300	241	191
	200	268	209	159	280	221	171	284	225	175
	300	258	199	150	270	211	162	274	215	166
9	100	224	183	146	234	193	155	237	196	159
	200	211	170	133	221	180	143	224	183	146
	300	204	163	126	213	172	135	217	176	138

TABLE 16.—*Stresses computed for the case of edge loading to show the effects of variations in the values of h, k, a, E and μ*

[P=10,000 pounds]

Thickness of slab, *h*	Modulus of subgrade reaction, *k*	Maximum load stress								
		E=3,000,000 *a* in inches—			*E*=5,000,000 *a* in inches—			*E*=6,000,000 *a* in inches—		
		2	6	10	2	6	10	2	6	10
Inches	*Lbs. in.*$^{-3}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$	*Lbs. in.*$^{-2}$
		μ=0.05								
6	100	685	468	319	719	502	352	730	514	364
	200	640	423	273	673	456	307	685	468	319
	300	613	396	247	646	430	280	658	442	292
8	100	389	295	217	407	314	236	414	321	242
	200	363	270	191	382	288	210	389	295	217
	300	348	255	176	367	274	195	374	280	202
9	100	307	242	183	322	257	198	327	262	203
	200	286	222	163	301	237	178	307	242	183
	300	275	210	151	290	225	166	295	230	171
		μ=0.15								
6	100	721	493	335	756	528	371	769	541	383
	200	673	445	287	708	480	323	721	493	336
	300	645	417	260	680	452	295	693	465	307
8	100	409	311	228	429	331	248	436	338	255
	200	382	284	201	402	304	221	409	311	228
	300	366	268	186	386	288	205	393	295	213
9	100	323	255	193	338	270	208	344	276	214
	200	301	233	171	317	249	187	323	255	193
	300	289	221	159	305	237	175	310	242	180
		μ=0.25								
6	100	757	518	352	794	555	389	807	568	402
	200	707	467	302	744	504	339	757	518	352
	300	677	438	273	714	475	310	728	488	323
8	100	429	326	240	450	347	261	458	354	268
	200	401	298	211	422	319	232	429	326	240
	300	385	281	195	405	302	216	413	310	223
9	100	339	268	202	355	284	219	361	290	225
	200	317	245	180	333	262	196	339	268	202
	300	304	232	167	320	249	183	326	254	189

SUPPLEMENTARY STUDIES

Mention was made early in this report of certain supplementary studies that were not a part of the originally scheduled program but which, because of circumstances, it seemed desirable to make and to report in conjunction with the work already described in this report. The supplementary studies comprise two distinct investigations; one, of the deflection and stress conditions in the vicinity of loaded slab corners and, two, of the stress reductions effected when the area over which the load is applied is considerably larger than any used in the tests described previously.

THE EFFECT OF LOADS ON SLAB CORNERS STUDIED

In the original manuscript of his paper "Theory of Stresses in Concrete Roads" Westergaard lists certain questions relating to the structural action of slab corners suggesting them as desirable subjects for further investigation. Among these were the following:

1. The influence of some nonuniformity in the distribution of the bending moment over the section of width $2x$; this influence may be expected to have the greater relative importance, the larger the value of the distance a_1, from the corner to the load.
2. The influence of twisting due to an eccentric position of the load, with the load closer to one edge than to the other.
3. The influence of an uneven thickness of slab, especially in the case of a thickened edge.

A fourth question, concerning impact loads, was mentioned but will not be discussed here. Certain data obtained in this investigation have a bearing on the three items that are listed, however.

The shape assumed by a slab corner when deflected by a load applied at or near the corner was determined by measuring the vertical displacements along the two edges of the slab corner and along the bisector of the corner angle.

In figure 45 deflection contours are shown for the 1st and the 60th application of a test load of 9,000 pounds on the corner of a slab of 7-inch uniform thickness, the loaded area being 8 inches in diameter and so placed that the slab edges were tangent to it. These data indicate that, while repeated loading caused a slight increase in the magnitude of the deflection at a given point, it produced no important change in the shape of the deflected corner area.[2]

Deflection contours for a similar symmetrical corner loading but of somewhat lesser magnitude are shown, for the 7-inch uniform-thickness and the 9–7–9-inch sections, in figure 46. It will be observed that within a distance of about 80 inches from the slab corner the

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[2] In this and in subsequent similar graphs the deflection contours are the result of averaging the measurements on the four free corners of the test section.

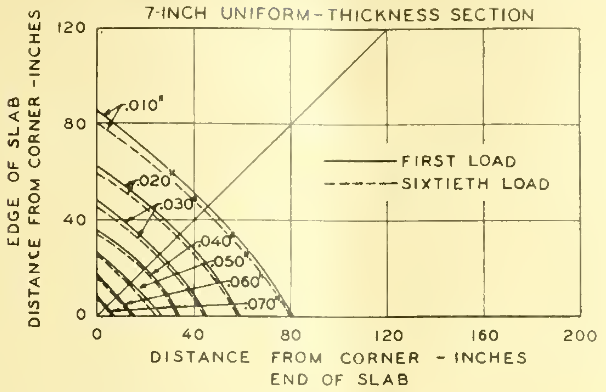

Figure 45.—Effect of Repeated Loading on the Deflected Shape of a Slab Corner—Symmetrical Loading.

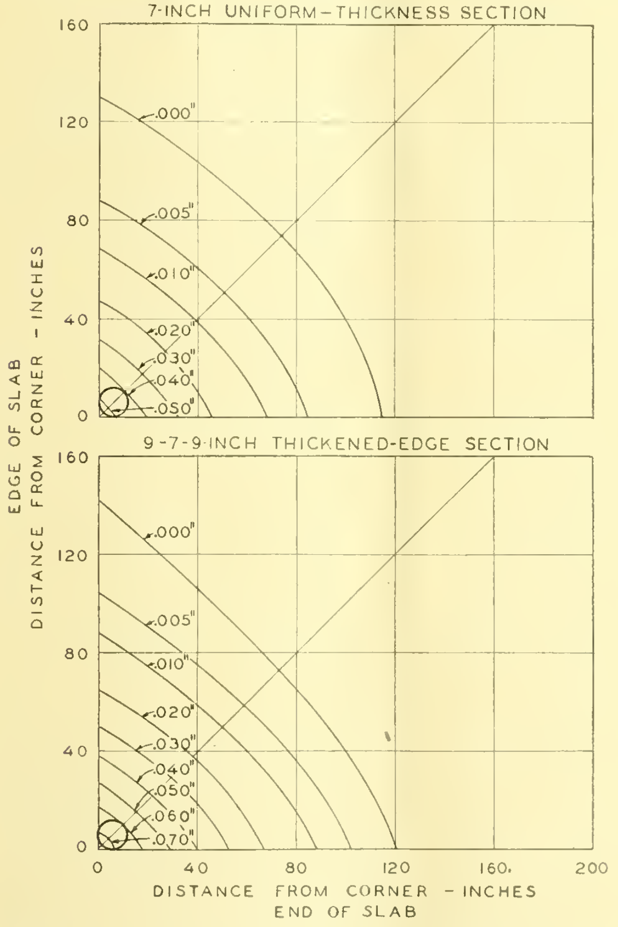

Figure 46.—Deflection Contours for Symmetrical Corner Loading.

deflections appear to be symmetrical on either side of the corner bisector for both the uniform-thickness and the thickened-edge cross sections. Beyond this distance the deflection of the slab end is slightly less than that of the side edge. The difference is small but appears in the data for both sections. It is believed to be due to the difference in the panel dimensions. Along the side edge there is a continuous structure 240 inches in length while along the end edge are two units each 120 inches in length hinged together at the longitudinal joint. It was stated earlier in the report that when the free corner is loaded there is a slight rotation about the longitudinal

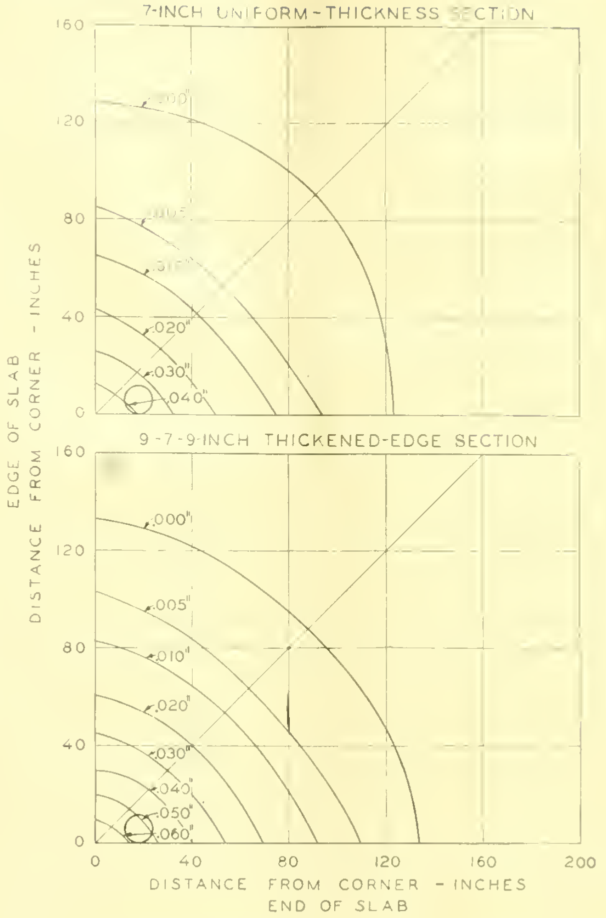

Figure 47.—Deflection Contours for Eccentric Corner Loading.

joint. Obviously there is no corresponding movement along the side edge of the slab.

In figure 47 are deflection contours for the same two corners under the action of a load of the same magnitude but displaced 18 inches toward the center line of the pavement. Such an eccentricity of load would tend to create a twisting moment in the slab corner. It is apparent from the data that with this loading the deflection, at a given distance from the corner, is greater at the end than at the side of the slab. A comparison of the data in figure 46 with those for the same test section in figure 47 shows that the maximum deflection for the given load is greater when that load is applied on the slab corner, which is as would be expected.

In order to determine both the direction and magnitude of the stresses at various locations in the corner region, measurements of strain were made in either three or four directions at each location. From these "rosette" strain measurements the direction and magnitude of the principal strains may be obtained by a method described by Osgood and Strum (*11*).

The location of the various gage lines used in this study were shown in figure 13 (quadrant 3). Where strains are measured in four directions instead of three, the fourth strain value serves as a useful check.

The installation of each strain rosette necessitated the drilling of eight small holes into which the gage points were cemented. Since these were installed in the tension face of the slab the tendency would be to weaken somewhat the flexural resistance of the corner and to cause some variation in relative strength at different distances from the corner. Because of this possibility certain of the tests were made with fewer installations than are shown in figure 13.

MAXIMUM STRAIN UNDER ECCENTRIC LOADING LOCATED

It will be noted that the strains were measured along the two slab edges and/or along three rays 22½° apart which quadrisect the corner angle, the center ray being the bisector of that angle. These rays will be spoken of as the corner bisector and the 22½° rays in the subsequent discussion. The positions of the loaded area in these special corner tests were described in connection with the deflection data just presented.

For the symmetrically placed corner loading on the 7-inch and 9-inch uniform-thickness sections, strains were measured with rosette installations along the 22½° rays and in two directions at intervals along the corner bisector. No strains were measured along the slab edges. For the same position of loading in testing the 8-inch uniform-thickness and 9–6–9-inch thickened-edge sections, strains were measured with rosette installations along both slab edges and the two 22½° rays and in two directions at intervals along the corner bisector. For this position of load, the direction of the principal strain in the region near the bisector of the corner angle was known to be parallel to that bisector.

For the eccentric corner loading, in which the position of the loaded area was moved along the end edge of the slab toward the pavement center line, for a distance of 18 inches, rosette installations were used along the two edges, the two 22½° rays and along the corner bisector as well.

The data obtained in these load-strain studies are shown in figures 48 to 66 inclusive.

Four of the test sections had loads applied symmetrically with respect to the slab corner, two had the load applied eccentrically. This made six combinations of load position and test section. For each of these six combinations there are three graphs.[3] The first graph in each group shows the direction of maximum strain for each of the 15 or more locations at which strains were measured. The second graph in each group shows the magnitudes of both the maximum strain and the minimum strain at each of these gage locations. The third graph of each group shows how the magnitude of the maximum strain varies with distance from the slab corner along the various rays on which gages were installed.

The slab corners used for the symmetrical loading were those of the 7-, 8-, and 9-inch uniform-thickness and the 9–6–9-inch thickened-edge sections. Those used for the eccentric loading were of the 8-inch uniform-thickness and the 9–6–9-inch thickened-edge sections.

The six graphs showing the direction of the maximum strains for the six slab-load combinations are figures 48, 51, 54, 57, 60, and 63, respectively.

The six graphs that show the magnitude of the maximum strains and the magnitude of the minimum strains (or those in the perpendicular direction at each gage location) are figures 49, 52, 55, 58, 61, and 64, respectively.

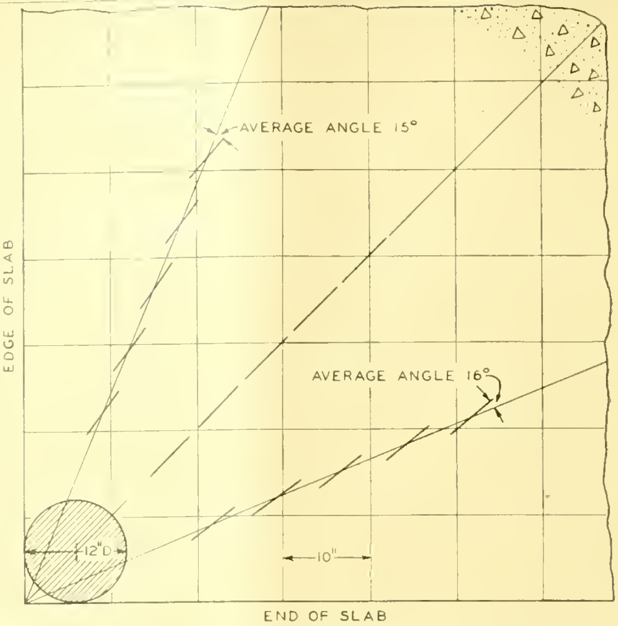

Figure 48.—Direction of Maximum Strains—7-Inch Uniform-Thickness Section—Symmetrical Corner Loading.

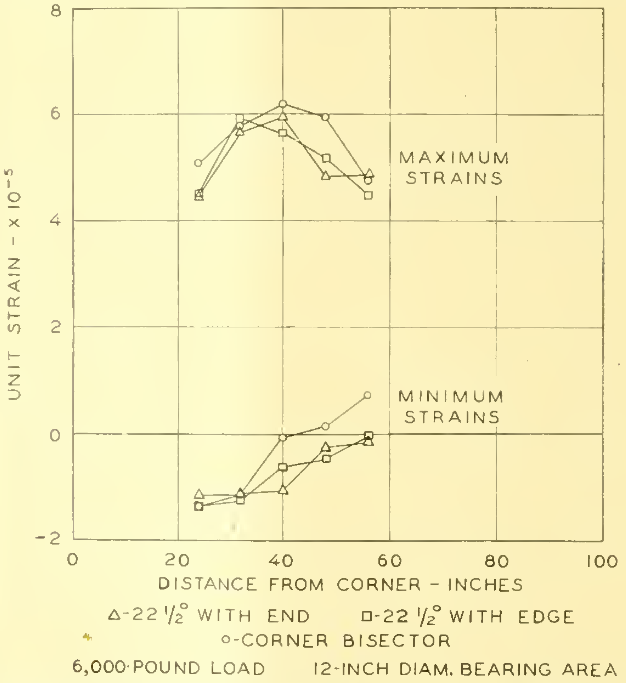

Figure 49.—Magnitude of Maximum and Minimum Strains at Each Gage Location—7-Inch Section—Symmetrical Loading—Average of Strains on Four Quadrants.

The six graphs showing the strain variation for various points along the rays on which the gages were installed are figures 50, 53, 56, 59, 62, and 65, respectively.

All of these figures are largely self-explanatory.

Referring to figures 48, 51, and 54 which show the direction of maximum strains for the symmetrical corner loading on the three sections of uniform thickness, it is apparent that while the maximum strain is along the central ray (the corner bisector) the maximum strains along the two 22½° rays are not parallel to the respective rays but have a direction which makes an angle of approximately 15° with these rays. The strains along the edges of the slab corner were not measured in the

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[3] In these figures and the attendant discussion maximum strain denotes the maximum strain for the given gage location and minimum strain is the strain measured at the same location in a direction perpendicular to that of the maximum strain. The maximum strains are approximately radial and the minimum strains approximately tangential in direction with respect to the center of load application.

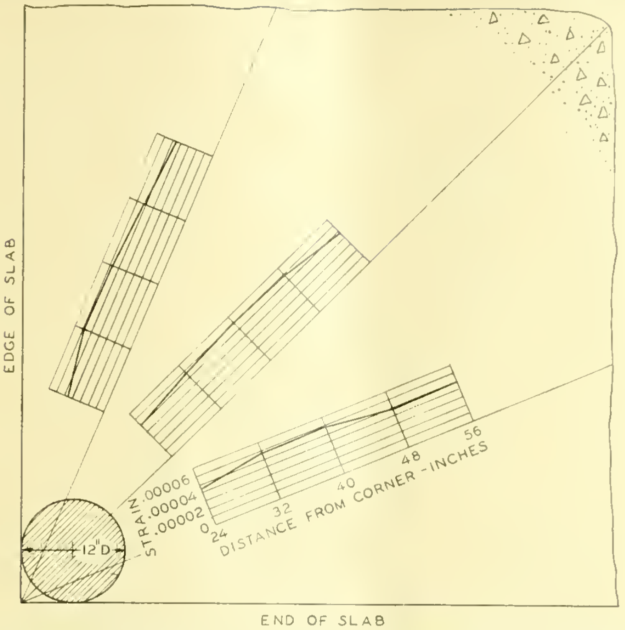


Figure 50.—Variation in Magnitude of Maximum Strains—7-Inch Uniform-Thickness Section—Symmetrical Corner Loading.

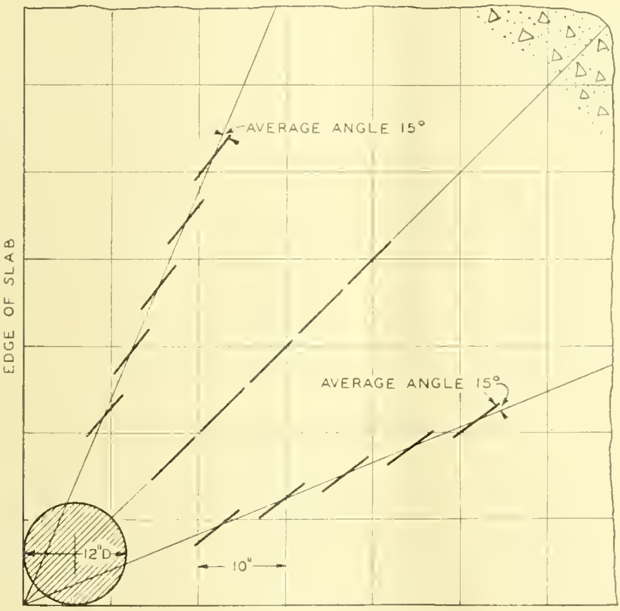


Figure 51.—Direction of Maximum Strains—9-Inch Uniform-Thickness Section—Symmetrical Corner Loading—Average of Strains on Three Quadrants Only.

cases of the 7-inch and 9-inch sections, figures 48 and 51. These strains were measured, however, in testing the corner of the 8-inch uniform-thickness section and, from figure 54, it is evident that the maximum strain makes an angle of approximately 28° with the slab edge.

For the symmetrical loading the directions of the maximum strains in the corner area of the 9-6-9-inch thickened-edge section were found to be essentially the same as for the uniform-thickness section (see figure 57). Apparently the thickening of the edge of this section was so distributed that it did not appreciably affect the stress trajectories.

The direction of the maximum strains changed appreciably, however, when the eccentric load, previously described, was used. The effect of this eccentricity can be seen by comparing figure 54 with figure 60 and figure 57 with figure 63. Because of the variation in the angle along some of the rays the range in the angle between the direction of the maximum stress and the ray is indicated by the small figures beside the rays in figures 57, 60, and 63.

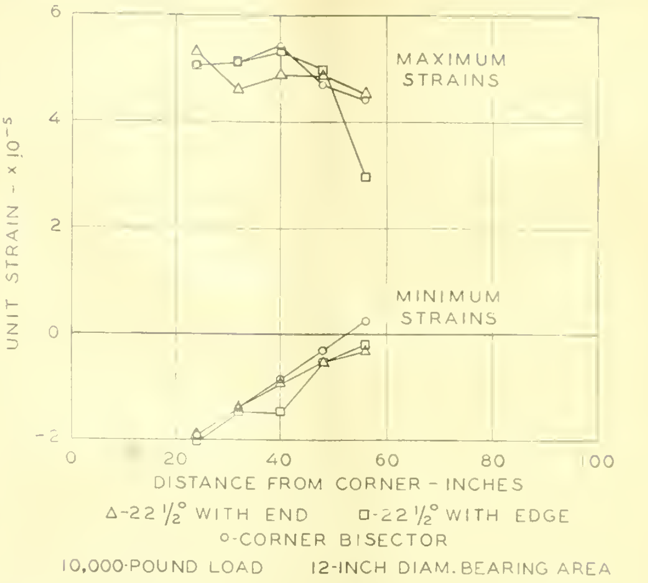


Figure 52.—Magnitude of Maximum and Minimum Strains at Each Gage Location—9-Inch Section—Symmetrical Loading—Average of Strains on Three Quadrants Only.

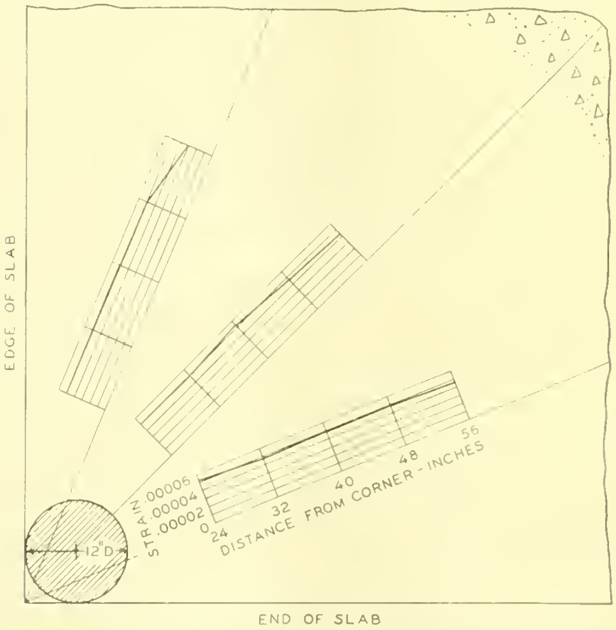


Figure 53.—Variation in Magnitude of Maximum Strains—9-Inch Uniform-Thickness Section—Symmetrical Corner Loading—Average of Strains on Three Quadrants Only.

SECTION OF MAXIMUM MOMENT NOT A STRAIGHT LINE

It was stated earlier that strains were not measured along the two slab edges in the cases of the 7-inch and 9-inch test sections and there was reason to believe that the installation of a large number of gage points in the tension face of the slab might affect the test data.

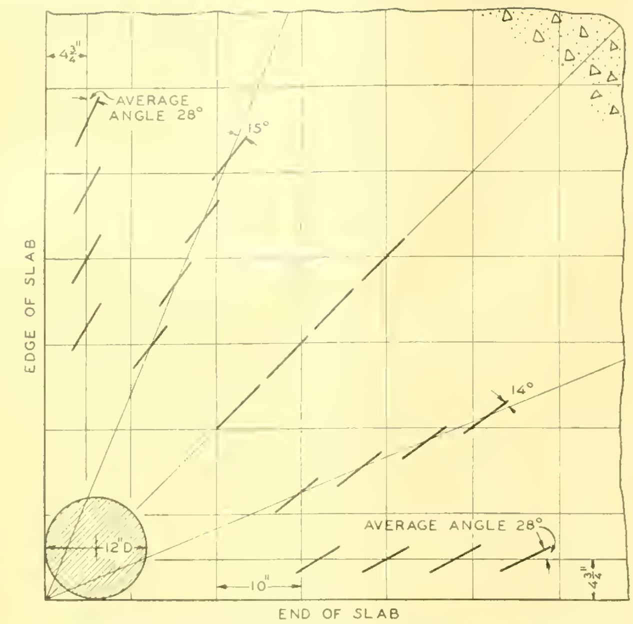


Figure 54.—Direction of Maximum Strains—8-Inch Uniform-Thickness Section—Symmetrical Corner Loading—Average of Strains on Three Quadrants Only.

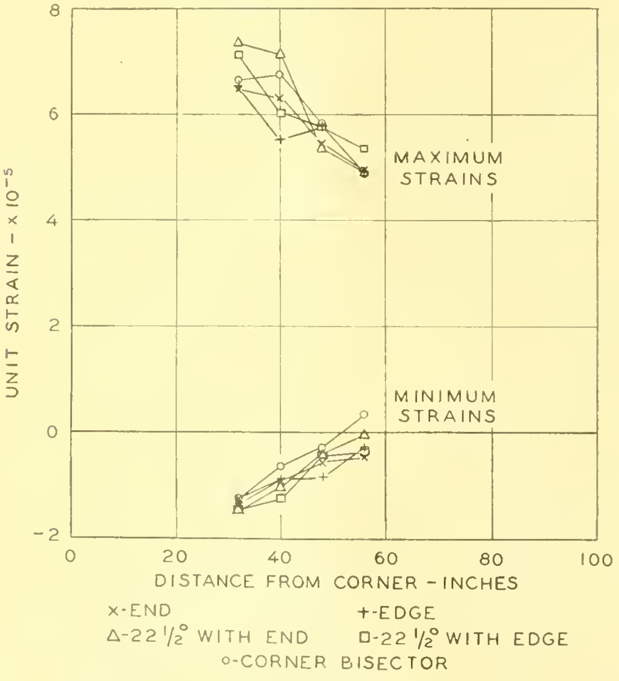


Figure 55.—Magnitude of Maximum and Minimum Strains at Each Gage Location—8-Inch Section—Symmetrical Loading—Average of Strains on Three Quadrants Only.

There is evidence to this effect in the set of graphs showing the strain variations along the several rays. The data for the 7-inch and 9-inch corners, figures 49, 50, 52, and 53, indicate the maximum measured strain to be on the corner bisector and at a distance of about 40 inches from the corner, for the conditions of the tests, while the maximum strain values on the two 22½° rays were of slightly lesser magnitude and at approximately the same distance from the slab corner. When, however, the strain gage points were installed along the two slab edges as well as on the rays mentioned it was found that the entire section of maximum strain was appreciably nearer the slab corner and the maximum strain no longer appeared on the corner bisector.

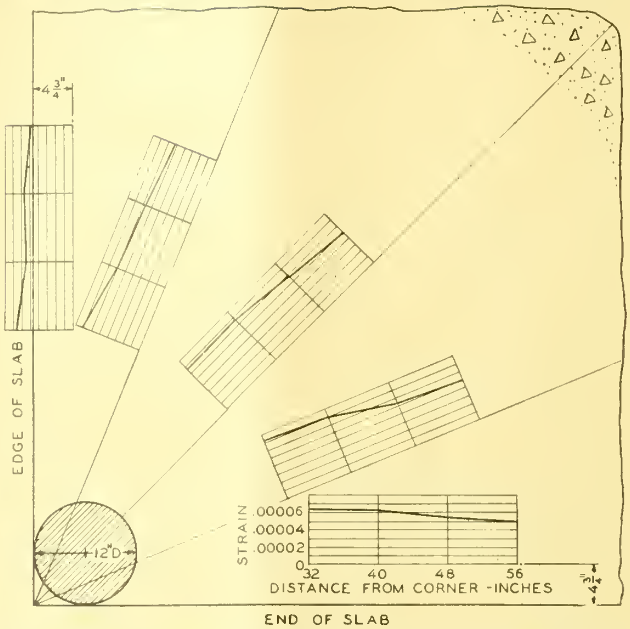


Figure 56.—Variation in Magnitude of Maximum Strains—8-Inch Uniform-Thickness Section—Symmetrical Corner Loading—Average of Strains on Three Quadrants Only.

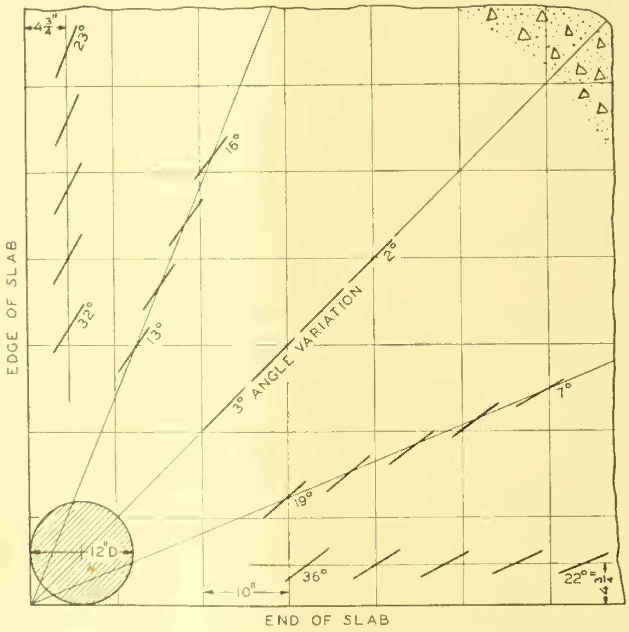


Figure 57.—Direction of Maximum Strains—9–6–9-Inch Thickened-Edge Section—Symmetrical Corner Loading—Average of Strains on Three Quadrants Only.

It is concluded, therefore, that the strain data for the 7-inch and 9-inch sections give a somewhat more reliable indication of the strain distribution in the corner of a section of uniform thickness than those obtained with the 8-inch-section and shown in figures 55 and 56.

Similar data for the eccentric loading applied to the 8-inch uniform-thickness and the 9–6–9–inch thickened-edge sections are shown in figures 61 and 62 and in figures 64 and 65, respectively.

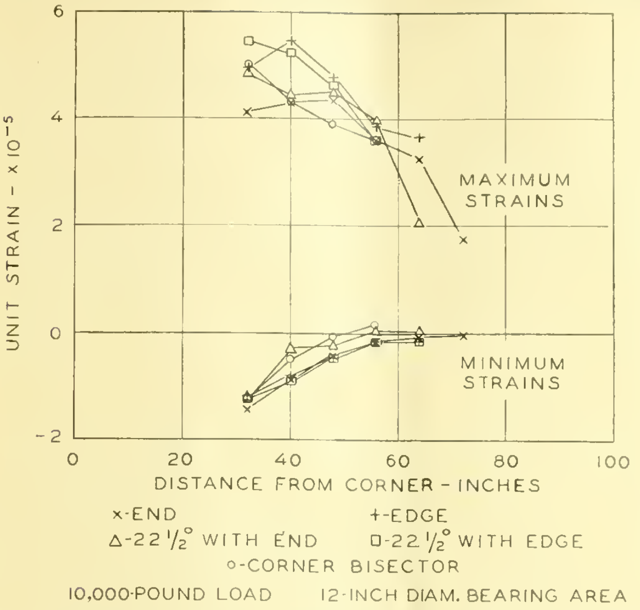


Figure 58.—Magnitude of Maximum and Minimum Strains at Each Gage Location—9-6-9-Inch Section—Symmetrical Loading—Average of Strains on Three Quadrants Only.

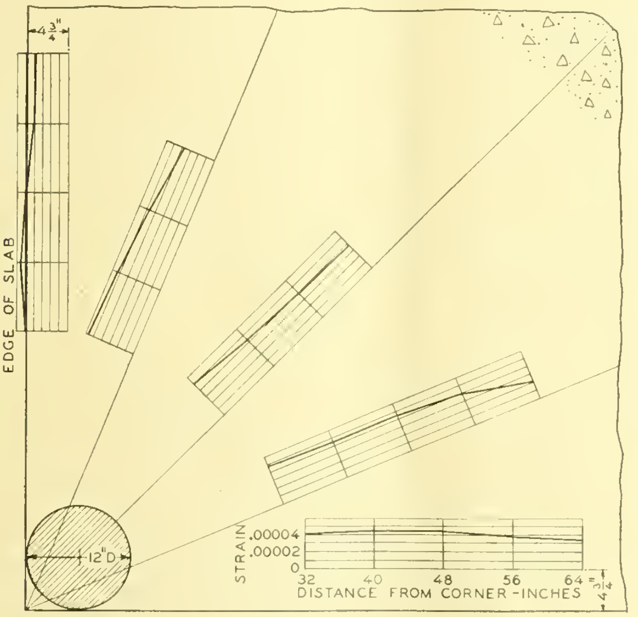


Figure 59.—Variation in Magnitude of Maximum Strains—9-6-9-Inch Thickened-Edge Section—Symmetrical Corner Loading—Average of Strains on Three Quadrants Only.

For a given test section there is a striking difference in the stress magnitudes resulting from the symmetrical and eccentric load positions used in this investigation. The maximum tensile strain caused by the symmetrical loading was approximately 50 percent greater than that caused by the eccentric loading and this applies to both the 8-inch uniform and the 9-6-9-inch test sections. Compressive strains, on the other hand, were higher for the eccentric loading.

It is apparent from the strain data which have been presented that there is a relatively large area across the slab corner within which the magnitude of the maximum stress, as previously defined, does not vary appreciably. This is in concordance with data obtained by Spangler and Lightburn (*14*) in carefully controlled laboratory studies of the strain distribution in the corner of a concrete slab.

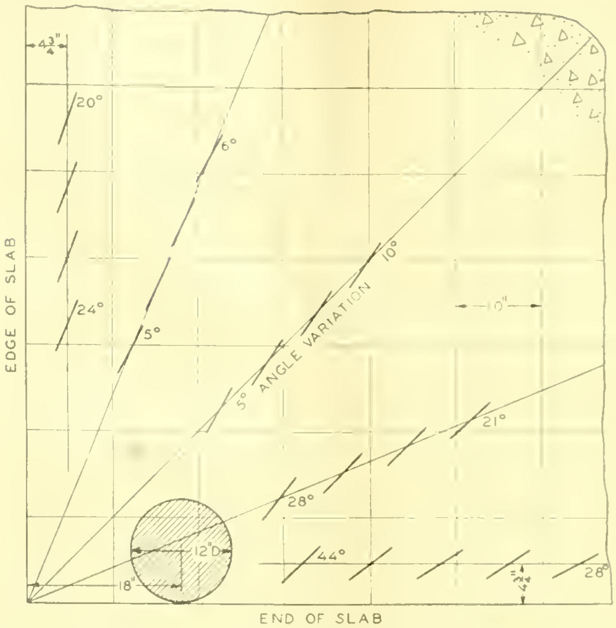


Figure 60.—Direction of Maximum Strains—8-Inch Uniform-Thickness Section—Eccentric Corner Loading—Average of Strains on Three Quadrants Only.

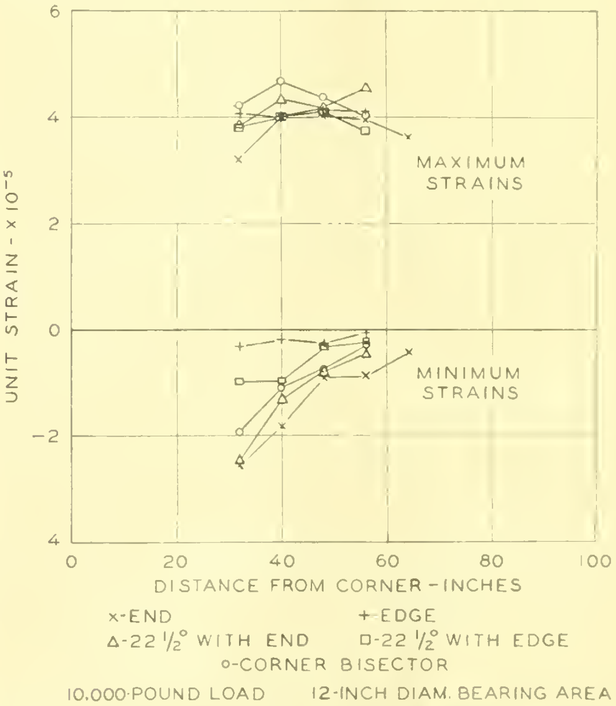


Figure 61.—Magnitude of Maximum and Minimum Strains at Each Gage Location—8-Inch Section—Eccentric Loading—Average of Strains on Three Quadrants Only.

In his original analysis of pavement slab stresses Westergaard states that in the case of corner loading when the distance x_1 (from the corner to the section of maximum moment) is not too large, the bending moment will be approximately uniformly distributed over the width, $2x_1$, of the cross section.

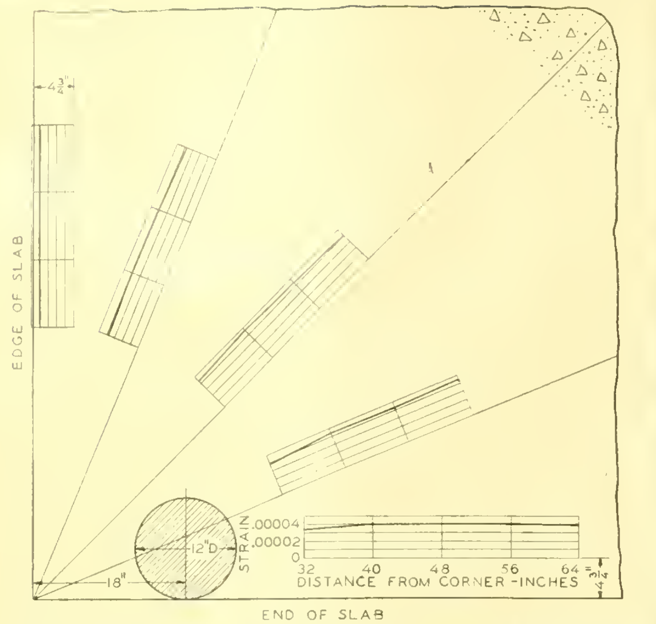


Figure 62.—Variation in Magnitude of Maxiumm Strains—8-Inch Uniform-Thickness Section—Eccentric Corner Loading—Average of Strains on Three Quadrants Only.

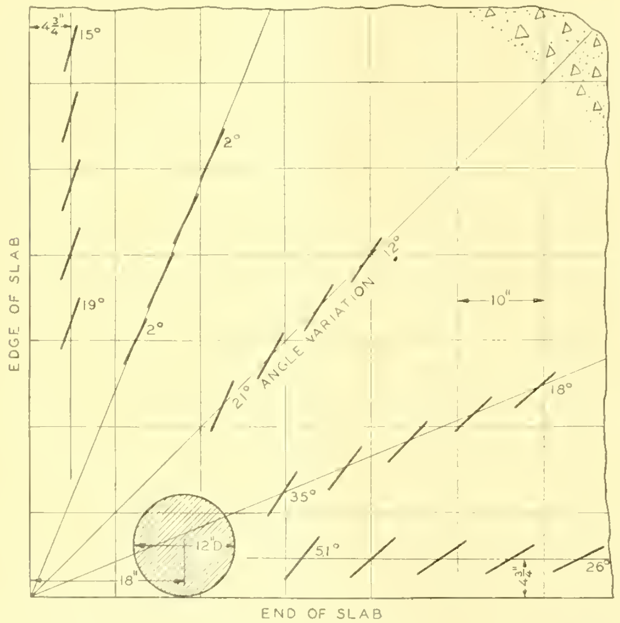


Figure 63.—Direction of Maximum Strains—9-6 9-Inch Thickened-Edge Section—Eccentric Corner Loading—Average of Strains on Three Quadrants Only.

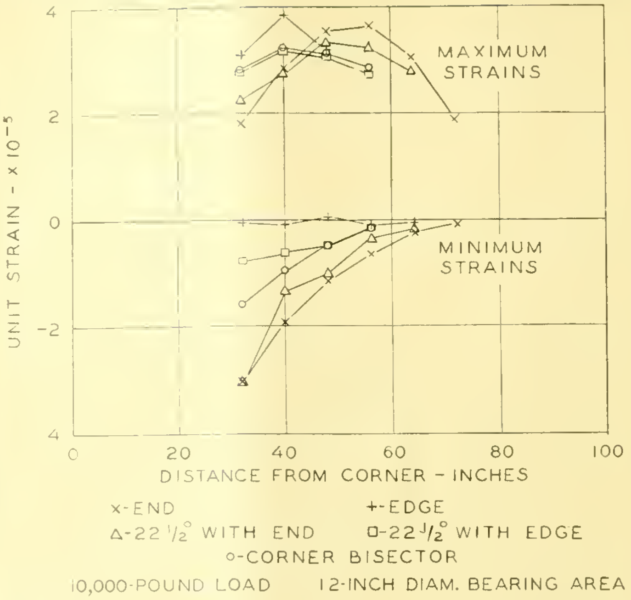


Figure 64.—Magnitude of Maximum and Minimum Strains at Each Gage Location—9-6-9-Inch Section—Eccentric Loading—Average of Strains on Three Quadrants Only.

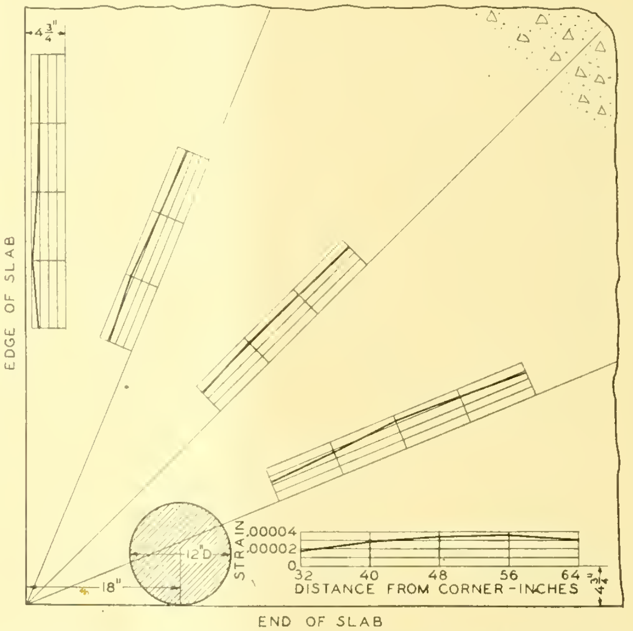


Figure 65.—Variation in Magnitude of Maximum Strains—9-6 9-Inch Thickened-Edge Section—Eccentric Corner Loading—Average of Strains on Three Quadrants Only.

Both observation of corner failures in the field and failures deliberately produced by corner loading during tests (*15, 22, 14*) have shown a tendency for the line of fracture resulting from a corner loading to be curved rather than straight although the curvature usually is not pronounced. The strain data that have been presented indicate that for the conditions of these tests the section of maximum moment is not a straight line normal to the bisector of the corner angle but follows a curved path similar to the deflection contours. In figure 66 are shown data obtained from the tests of the 8-inch uniform-thickness section. In this figure maximum strain values, as determined from strains measured in four directions, have been resolved to show the component in the direction parallel to the corner bisector. These are the strain values on the straight line section across the corner perpendicular to the corner bisector. In the figure the section shown is at a calculated distance, $x_1=32.2$ inches, from the corner. In addition to the plotted values of the strains observed on the five lines radiating from the slab corner the value of the average normal strain is shown by the horizontal dash line. It is apparent that in this test the component of the maximum strain observed on one of the 22½° rays was approximately 10 percent greater than the average

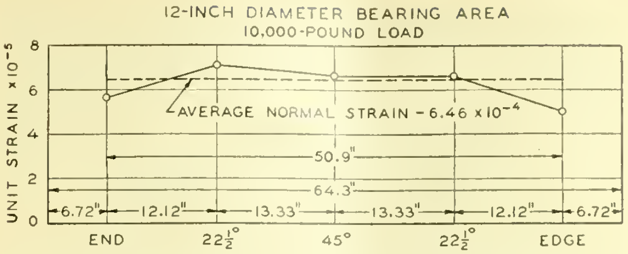


FIGURE 66.—DISTRIBUTION OF OBSERVED STRAINS ACROSS PLANE SECTION PERPENDICULAR TO CORNER BISECTOR AT A COMPUTED DISTANCE, X_1, FROM THE CORNER—8-INCH SECTION—SYMMETRICAL LOADING—AVERAGE OF STRAINS ON THREE QUADRANTS ONLY.

strain normal to the section. The variation in strain across the assumed plane section is evident.

Since strains were not measured along the slab edges at the corners of the 7-inch and 9-inch sections it is not possible to analyze the data from those sections in the manner shown in figure 66.

LOAD TESTS MADE WITH LARGE CONTACT AREAS

Originally the Westergaard analysis of the stresses in concrete pavement slabs (*23, 25*) was a discussion of the effects of such loadings as were being encountered in highway practice and, for purposes of simplicity, it was assumed that the load was applied to the pavement over areas that were either circular or semicircular in shape.

In planning the program of tests previously discussed in this report, provision was made for a range of circular and semicircular areas of contact sufficient to represent the tire contact areas normally met with in highway service.

Subsequently Westergaard supplemented his original analysis with two papers which discussed respectively the effect of (1) larger wheel loads on correspondingly larger loaded areas (*27*) and (2) loaded areas that are of other than circular shape (*28*).

In the first paper (*27*) the analysis for the case of interior loading is reexamined to determine its suitability for determining stresses when the areas of contact are much larger than those found in highway service. It was concluded by the author that the analysis is applicable to the conditions of loading found in airport runway service except that under some conditions certain small corrections are desirable. The means for making these corrections are given in the paper. Only the interior case of loading is discussed.

In the second paper (*28*) the effect of shape of loaded area is one of the subjects discussed. This is a valuable addition to the analysis since it furnishes for the first time the means for estimating the importance of variations in shape of loaded area which are known to exist. Both of these subjects are of great current interest because of their direct bearing on the design of concrete pavement slabs for airport service.

The original program of the investigation being reported contained no provision for tests with very large areas of contact since it was intended to cover the conditions of highway service. However, before it became necessary to vacate the test site there was an opportunity to make a limited number of tests that yielded data which afford significant comparisons between theory and observed performance for concrete pavements loaded over relatively large areas of both circular and elliptical shape.

The concrete pavement available for the load tests with the large contact areas was one which had been constructed in 1940 for use in another investigation. It was 20 feet wide divided at the center by a tongue and groove joint with ⅝-inch diameter tie-bars at 60-inch intervals. The sections were 30 feet in length between transverse expansion joints and were divided into 15-foot slab lengths with contraction joints of the plane of weakness type. The slab units were thus 10 by 15 feet. The pavement was of 8-inch uniform depth.

The concrete used in the pavement was mixed in the following proportions (dry weight):

	Pounds
Cement	94
Fine aggregate (sand)	200
Coarse aggregate (siliceous gravel)	330

The concrete had a 2½-inch slump at the time of placing. At 28 days the average crushing strength was 4,600 pounds per square inch and the average modulus of rupture was 550 pounds per square inch. The average modulus of elasticity at the time of the actual load testing on the pavement was 4,800,000 pounds per square inch.

These test sections had been laid on the site of some of the sections of the original investigation (*18*). However, the grade had been changed slightly and the character of the subgrade immediately under the new sections had been altered by mixing sand with the original silty loam to a depth of several inches and recompacting at optimum moisture to maximum density. This resulted in an average dry density of 136 pounds per cubic foot within the treated depth. The effect of this treatment will be discussed later.

The equipment used for applying the loads was essentially the same as that described in the first report of this series except that for the larger loads a hydraulic jack and test gage were substituted for the mechanical jack and steel beam dynamometer. The hydraulic jack and test gage were calibrated prior to use in the tests.

Loads were applied to the pavement through rigid bearing plates that were either circular or elliptical in shape. The elliptical plates were of the proportions shown in figure 67. This is a conventionalized average obtained from actual tire impressions which seem to conform reasonably well to the contact areas of large-size, low-pressure pneumatic tires at recommended inflation pressure and capacity load. Dimensional data for both the circular and the elliptical areas of contact are given in table 17.

TABLE 17.—*Dimensions of contact areas*

Circular contact areas		Elliptical contact areas		
Diameter	Area	Length of major axis	Length of minor axis	Area
Inches	*Sq. ins.*	*Inches*	*Inches*	*Sq. ins.*
8	50	12	6.86	72
12	113	24	13.71	289
20	314	36	20.57	649
25	491	48	27.43	1,155
30	707			
36	1,018			

Considerable care was taken in seating the bearing plates to insure uniform distribution of the load over the area of contact. A thin layer of plaster of Paris was first placed on the concrete test slab and formed to

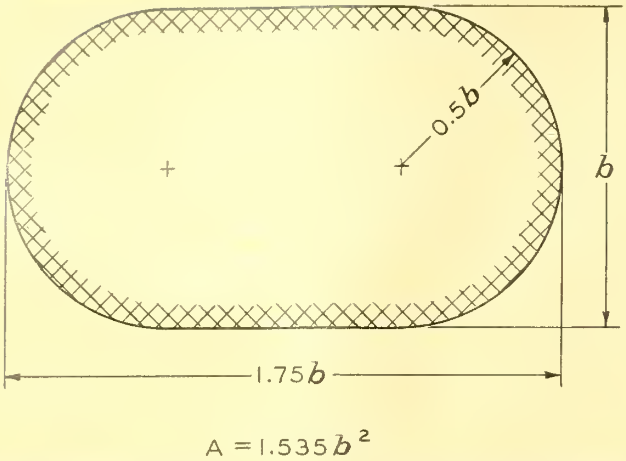


Figure 67.—Proportions of Elliptically Shaped Bearing Areas.

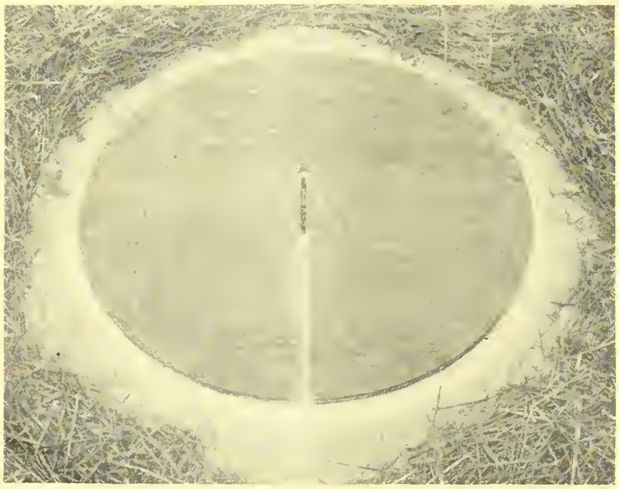


Figure 68.—Circular Plywood Bearing Plate Seated on the Pavement.

a smooth plane surface. When this had hardened, a sponge rubber mat ½ inch in thickness was placed on the plaster surface. A plate of plywood, 1 inch in thickness, cut to the exact shape of the desired area of contact was then laid on the rubber mat and on this plywood plate an assembly of stiff plates was built up in such a manner as to insure rigidity. These back-up plates were of steel and of concrete and were bedded individually in plaster of Paris. Typical circular and elliptical plywood plates are shown in figures 68 and 69 respectively, while the complete assembly for a test with a circular area is shown on the cover page.

TEST PROGRAM PLANNED TO YIELD DATA FOR AIRPORT DESIGN

Strains were measured with the recording type of strain gage used throughout the investigation except that for these tests the glass slide on which the movement of the stylus is traced was fitted into a small carriage so arranged that it could slide longitudinally on the gage body. This carriage was moved by means of a fine wire attached to it and extending from the gage position to the edge of the loaded area in a groove provided in the plywood plate. The strain gage, groove, and wire can be seen in figures 68 and 69. This arrangement permitted a series of strain measurements to be made without disturbing the bearing plate assembly.

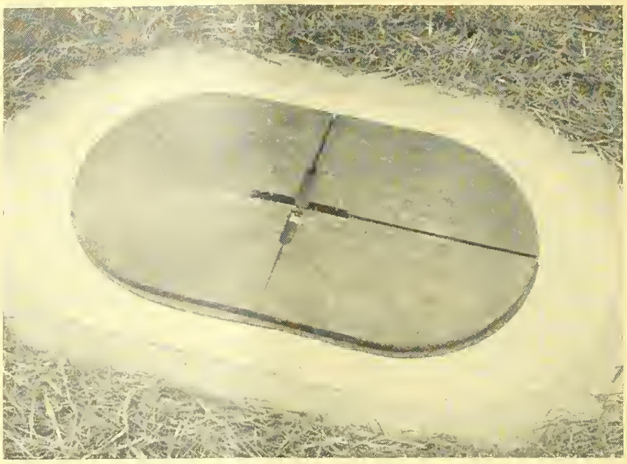
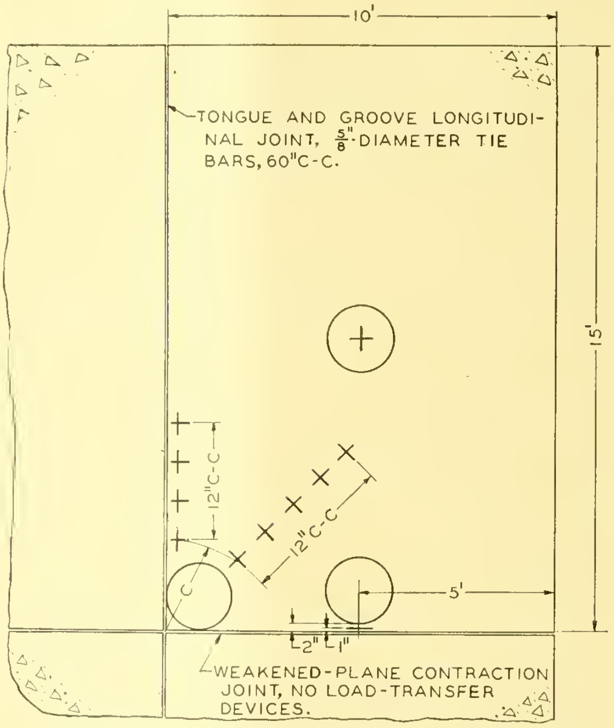


Figure 69.—Elliptical Plywood Bearing Plate Seated on the Pavement.


10'
TONGUE AND GROOVE LONGITUDINAL JOINT, ⅝"-DIAMETER TIE BARS, 60"C-C.
15'
12"C-C
12"C-C
C
5'
2"
1"
WEAKENED-PLANE CONTRACTION JOINT, NO LOAD-TRANSFER DEVICES.

THE DISTANCE, C, RANGED BETWEEN 16 INCHES FOR THE 8-INCH-DIAMETER BEARING AREA AND 40-INCHES FOR THE 36-INCH DIAMETER BEARING AREA.

Figure 70.—Locations at Which Loads Were Applied and the Positions at Which Gages Were Placed to Measure the Critical Strains.

Because of a limit on the time during which the pavement sections were available it was necessary to restrict the study to a comparatively small number of tests. It was decided, therefore, to apply loads at the locations that seemed most important from the standpoint of airport pavement service. The locations selected were:

1. An interior corner, i. e., one formed by the intersection of a transverse contraction joint and the longitudinal joint.
2. An interior point, as far as possible from slab edges.
3. A transverse contraction joint edge at some distance from a slab corner.

These locations together with the strain gage positions used with each are shown in figure 70.

No tests were made at free edges or free corners and no tests were made along the longitudinal joint edge because it had been found in earlier tests (*21*) that the type of joint in these slabs is highly effective in controlling the critical stress caused by a load acting in the region along the joint.

A study was made of the strain distribution within these large bearing areas for the interior and edge loadings and it was found that strains measured at the positions shown would be as high as any within the area. In the case of the corner loading where strains were being measured outside the bearing area it was necessary to make preliminary tests to locate the position at which the strain along the bisector of the corner angle would be a maximum for each of the several bearing plate sizes. In this connection it may be recalled that the studies of joint behavior described in the fourth report of this series (*21*) showed that at interior slab corners, where corner support is obtained from adjoining panels, the critical stress may develop directly under the loaded area or at some point along the edges of the loaded panel rather than along the bisector of the corner angle as is the case with a free corner.

The protection given the test section and other details of procedure were the same as those already described for the main part of the investigation.

SUBGRADE TESTS GIVE SURPRISING RESULTS

Since the character of the subgrade immediately beneath the pavement of the test section had been altered in the manner previously described it was necessary to determine a value for the effective modulus of subgrade reaction before any comparisons could be made between theoretical and observed stresses. These subgrade tests developed data that are of considerable interest.

It was found in the determination of values for k on the original subgrade that a rigid circular plate 30 inches or more in diameter was of sufficient size to eliminate the effect of plate size. Because of this finding the first tests on the modified subgrade were made with a rigid circular plate of 36-inch diameter in the manner described in the first section of this report. The data obtained indicated a value of approximately 400 lbs. in.$^{-3}$ for the modulus of subgrade reaction and, contrary to the data from the earlier tests (see fig. 12), showed about the same values for plate displacements of 0.01, 0.02, and 0.05 inch. While it appeared reasonable to expect that the treatment given the subgrade soil would cause an increase in its resistance to deformation under load, it had not been anticipated that the rather marked effect of the magnitude of the plate displacement on the modulus of subgrade reaction which had been evident in the bearing tests on the original subgrade would be so greatly reduced in the tests on the modified subgrade.

The value of the modulus, k, as determined from load deflection tests on the pavement on one quadrant of the test section was found to be 285 lbs. in.$^{-3}$, considerably lower than that indicated by the bearing tests with the 36-inch diameter rigid plate but essentially the same as that found for the original unmodified subgrade under similar summer conditions.

This rather surprising result has several interesting implications. It is indicated that in modifying the character of the upper layer of the subgrade, in the manner previously described, its load supporting ability within a given deformation limit was considerably increased so long as the given unit load was applied over a relatively small area. When the loaded area was relatively large, as with the slab deflection test, on the other hand, the influence of the strengthened upper layer on the load support offered by the subgrade as a whole tended to disappear. This suggests that there is a relation between the size of a loaded area on a pavement and the strengthening effect to be expected from a given base course beneath that pavement.

Unfortunately there was no opportunity to study this matter thoroughly in connection with the present investigation. It was possible, however, to make determinations of the value of k from slab deflection tests on other quadrants of the test section and also to make one determination with a rigid bearing plate of greater size. This latter test was made with a 54-inch diameter plate on the subgrade in an opening cut through the pavement in one quadrant of the test section. This is shown in figure 71. The data obtained from the bearing test indicated a value of $k = 315$ lbs. in.$^{-3}$ at a displacement of 0.02 inch, about 10 percent greater than the value obtained from the slab deflection in this particular quadrant. This difference in the values of the modulus of subgrade reaction would have a negligible effect on the magnitude of computed stresses for the conditions that obtained in this study.

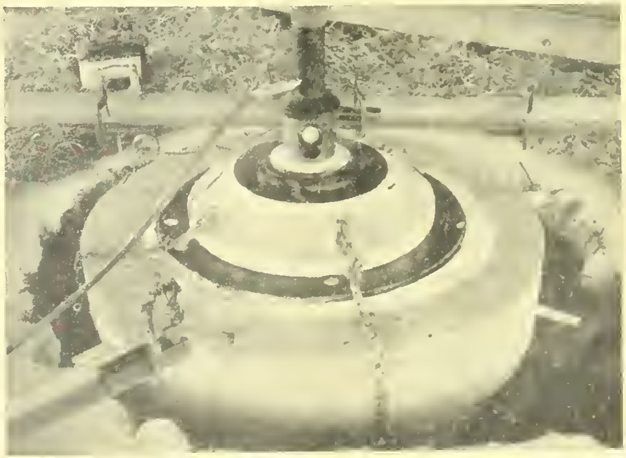


Figure 71.—Determining the Modulus of Subgrade Reaction With a Large Bearing Plate Placed on the Subgrade Through an Opening Cut in the Pavement.

It was not possible to make bearing tests with the 54-inch diameter rigid plate through the pavement on each quadrant of the test section. An average value for k determined in this manner is, therefore, not available.

The average value obtained from the data obtained in the load-deflection tests on the pavement in three of the four quadrants of the test section was 250 lbs. in.$^{-3}$. This value was used in computing certain of the stress values that will be shown in one of the subsequent graphs.

In order to obtain data that would permit strain data to be handled more flexibly in the analysis, a study was made of the linearity of the load-strain relation for a range of areas of contact which included both those used in the general investigation and the larger areas used in this supplementary study. These tests were made for interior loading only. Data obtained with circular contact areas with diameters ranging from 8 to

36 inches are shown in figure 72. The maximum load magnitude was sufficient in each case to produce a stress of at least 300 pounds per square inch. Each value shown in this figure is the average of eight observations, two in each of the four quadrants of the test section.

From these data it can be concluded that the load-stress relation is linear within the stress range shown for all the contact areas used.

UNCERTAIN NATURE OF EDGE SUPPORT AT FRACTURED FACES DEMONSTRATED

In figure 73 there is shown a comparison of stress values computed by means of the Westergaard equation with stress values derived from the strains observed in the loading tests, for each of the six sizes of circular contact area included in the tests.

For concrete slabs lying on a subgrade and acted upon by vertical static loads applied in the interior region, both theory and test show that the maximum flexural stress is closely proportional to the applied load. This fact makes it possible to extend load-stress data beyond the range of observed values without the risk of appreciable error. In deriving the observed stress values of figure 73 the data were extended somewhat beyond the observed range in the cases of the two smaller bearing areas in order to obtain stress values corresponding to the assumed load of 20,000 pounds. While these two stress values may be slightly above the usual working stress range for concrete in flexure the procedure is sound and the comparison between the theoretical and observed values valid throughout the stress range shown.

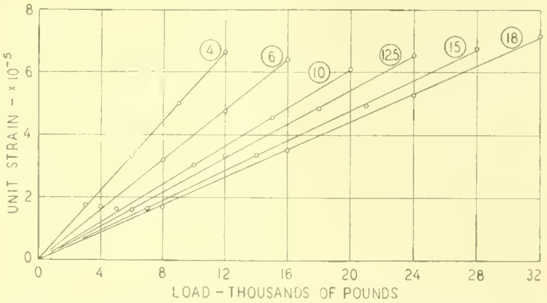
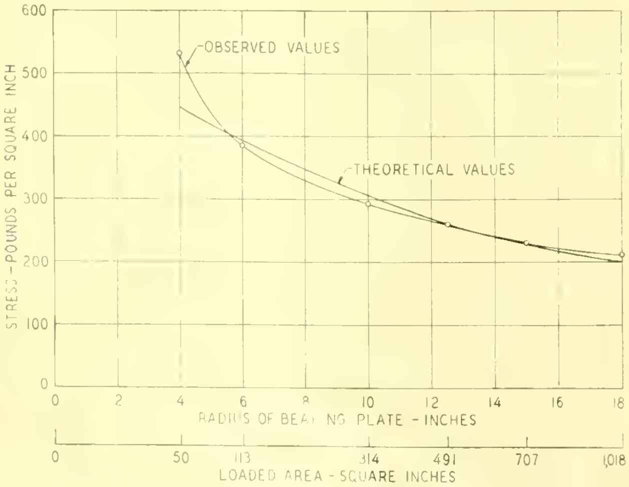


Figure 72.—Load-Strain Relation for a Range of Size of Loaded Circular Areas—Interior Case of Loading—Figures in Circles Show Radii of Loaded Areas.



CONSTANTS.- P=20,000 POUNDS h=8 INCHES E=4,800,000 LB. INCH^{-2}
μ=0.15 k=250 LB. INCH^{-3} Z=0

Figure 73.—Comparison of Theoretical and Observed Stresses for a Range of Size of Loaded Circular Areas—Interior Case of Loading.

It will be noted in the legend that, for calculating the theoretical stresses, it was assumed the quantity Z (termed the ratio of reduction) equals 0. When this value is assumed the general agreement between the theoretical stress values and those obtained from the measured strains is remarkably close and it is evident that the observed effect of size of loaded area on the stress developed by a given load is essentially the same as the theoretical relation. The one exception is the value obtained with the 8-inch diameter area where the observed value exceeds the theoretical by approximately 19 percent. The reason for this difference is not known. Referring back to similar data from the tests on the original pavement sections, shown in figure 40, it will be observed that the same tendency is evident where the 8-inch diameter area was used on the 8-inch and 9-inch sections of uniform thickness. In this connection it is pointed out that a contact area of this size is usually associated with a wheel load of about 3,500 pounds which, with pavement slabs of usual thicknesses, would cause a stress of less than 100 pounds per square inch. Thus the disparity occurs under conditions that are relatively unimportant.

In the computations for the theoretical stress values of figure 73 a value of $k=250$ lbs. in.$^{-3}$ was used since this represented an average from tests on three quadrants. While this value may appear somewhat low in view of the data from the bearing tests, it is worth noting that the use of a value of $k=300$ lbs. in.$^{-3}$ would cause a decrease in the computed stress values of only about 3 percent.

From these data it may be concluded that for the conditions of these tests there is good agreement between the observed stresses and those computed by means of the Westergaard equation for bearing areas of circular shape over a range of diameters from 8 to 36 inches.

Tests were made also with loaded areas of elliptical shape. On the basis of strain distribution studies the gages were located at the center of both the major and minor axes of the area as shown in figure 69. In figure 74 the stress values derived from the strains measured along each axis are shown for all of the elliptical areas listed in table 17, i. e., for a range of areas from 72 to 1,155 square inches. The observed maximum stresses for corresponding circular areas are shown on the same graph. The graph affords several useful comparisons. In the first place it is apparent that the stress along the major axis of the elliptical area is less than that along the minor axis, the difference being as much as 20 percent for the largest area. The maximum stress observed under each corresponding circular area was somewhat greater than that found along the major axis and slightly less than that found along the minor axis of the elliptical area.

In the past, for purposes of stress computation for highway loadings, it has often been assumed that the actual tire contact area could be represented by the circle of equivalent area. It is interesting to note that for areas such as are found in highway service, the data indicate the error resulting from this assumption would be small, probably 5 percent or less. For the larger areas the percentage would be slightly more.

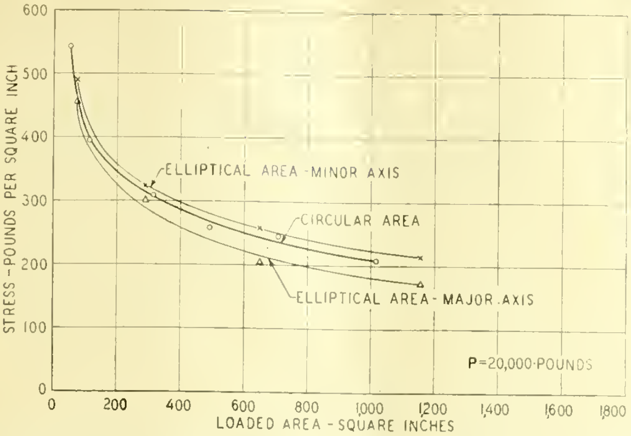


Figure 74.—Comparison Between Maximum Stresses Observed Under Circular and Elliptical Bearing Plates of Equal Area—Interior Case of Loading.

It was mentioned earlier that Westergaard has, in a recent paper, extended his analysis to include loaded areas of other than circular shape (*28*). In a discussion of a part of this paper by the authors (*28*) comparisons were made between observed stresses and those computed by the equations developed by Westergaard for elliptically shaped loaded areas. It was shown that there was good agreement between the observed data and the theoretical relations.

Figure 75 shows observed stress values for a load of 20,000 pounds applied on circular contact areas ranging from 8 to 36 inches in diameter placed at each of three load positions shown in figure 70. When the load was applied at the mid-point of the slab end, i. e., along the weakened plane contraction joint midway between the slab edge and the longitudinal joint, tests were made with the contraction joint open and with it closed by a horizontal force of 60,000 pounds (over the 10-foot lane width) applied at the ends of the test section. When the load was applied at the interior corner the contraction joint was open.

The strain gages for the interior corner loading were located along the bisector of the corner angle at the position to measure the maximum strain developed in this direction. It is evident from the relatively low stress values that the slab corner is being strongly supported along the longitudinal joint and does not act as a free corner. Rather it approaches the condition of a free edge and the maximum stress for this case would be within the loaded area and more nearly equal in magnitude to that found in the test at the mid-point of the slab end, with the joint open.

The stress values for the interior loading serve as a reference in analyzing the other stress values.

The stresses observed for the load applied at the mid-point of the slab end bear a normal relation to those for the interior loading so long as the contraction joint is open. When, however, the joint is closed by pressure applied at the opposite ends of the test section, it is observed that the stresses for this edge loading are reduced appreciably for all sizes of bearing area and that for the larger areas the stresses become essentially equal to those found in the tests at the interior of the slab. For convenience the stresses, expressed as the percentages by which they exceed the stress for interior loading, are shown at three points along the graph. This comparison shows very clearly the uncertain nature of the edge support furnished by the fractured faces alone in a joint of the plane of weakness type, and indicates the need for edge strengthening if a balanced design is to be obtained.

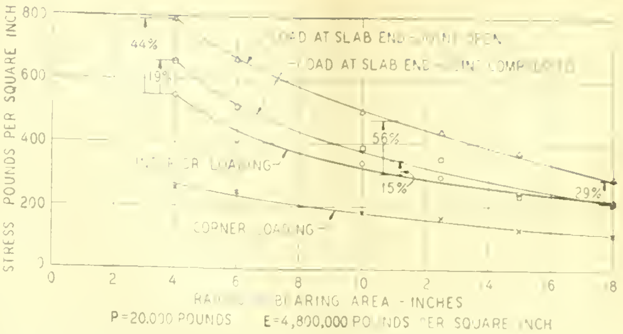


Figure 75.—Effect of Variation in Position of Load and Size of Bearing Area on the Stress Caused by a Given Load.

CONCLUSION

In the presentation and discussion of the data obtained in this rather extended study, an effort has been made to simplify the presentation by dividing the material into sections, each more or less complete in itself, with occasional summary statements in which the significant developments are pointed out and such conclusions as seem warranted are given. These detailed statements need not be repeated here.

The broad purpose of all of the studies described in this report has been to compare the observed structural behavior of pavement slabs of uniform thickness supported by a subgrade and subjected to vertically applied static loads with the behavior indicated by the Westergaard analysis (*23*, *25*, *27*, *28*). On this point it is concluded that, within the limits of the investigation and so long as the basic conditions assumed for the analysis are approximated, the Westergaard theory describes quite accurately the action of the pavement.

There is need for further experimental study of some of the quantities that appear in the analysis, however. It is particularly desirable that information be developed which will permit the equations to be applied more readily and with greater certainty to problems that require the computation of values of absolute stress.

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D. M. BEACH, *Editor*

Volume 23, No. 9 July–August–September, 1943

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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AMOUNT AND CHARACTERISTICS OF TRUCKING ON RURAL ROADS

BY THE DIVISION OF HIGHWAY TRANSPORT, PUBLIC ROADS ADMINISTRATION

Reported by John T. Lynch, Senior Highway Economist and Thomas B. Dimmick, Associate Highway Engineer-Economist

Our highways play a vital role in the movement of goods, both in war and in peace, but because the trucking industry is made up mainly of numerous small carriers, statistical reports such as those compiled periodically for railways have not been made. The highway planning surveys, however, have afforded the means of estimating the amount and characteristics of trucking on rural roads.

This report presents estimates of vehicle-mileage, percent loaded, average carried load, and ton-mileage for trucks on main and local rural roads in 1940, and on main rural roads in 1942, prepared on the basis of data obtained principally in the initial and continuing phases of the highway planning surveys. Separate estimates are made for single-unit trucks and for truck combinations. The 1940 estimates are subdivided to show the nature of the haul as regards origin and destination, classified as rural or urban, and as within or without the State under consideration. For 16 States, analysis is made of trip extent on the basis of the number of counties traversed. The frequency of heavy gross weights and axle loads in 1940 and 1942, and the frequency of heavy load concentrations in 1942 are also presented.

Freight hauling by truck on rural roads in 1940 amounted to about 59 billion ton-miles—47 billion on main roads, and 12 billion on local roads. City-to-city hauling made up slightly more than half of this ton-mileage. Total rural hauling was about 68 percent intrastate, and only about 32 percent was interstate or transstate.

Truck vehicle-mileage fell off in 1942, but heavier loads resulted in approximately the same ton-mileage on main roads as in 1940. Use of truck combinations has increased greatly.

Frequency of both heavy gross loads and heavy axle loads increased from 1940 to 1942. Heavy gross loads were most frequent in the Pacific region, but heavy axle loads were most frequent in New England. Interior groups of axles were found to produce excessive load concentrations more frequently than the over-all wheel base.

LARGE VOLUMES of freight have moved over our highways for a number of years, but little has been known until recently concerning the amount and characteristics of this movement. The nature of the trucking industry makes impracticable the periodic reporting of comprehensive statistics such as those which have long been compiled for railroads. It is true that the large interstate common and contract carriers, having annual gross operating revenues in excess of $100,000, make extensive reports to the Interstate Commerce Commission, but these carriers constitute less than one percent of total truck operators. There are almost 3,000,000 truck owners in the United States, about 85 percent of whom own only one truck, and few of whom keep records useful as a basis for statistical reports. A truck and bus inventory, which will yield certain statistical data concerning vehicle types and annual mileages driven, is now being made as part of the program of the Highway Traffic Advisory Committee to the War Department, but this will not give information on the tonnages transported by truck or on the origins and destinations of trips.

HIGHWAY PLANNING SURVEY SUPPLIES BASIC DATA

The vital role of our highways in the movement of goods, both in war and in peace, makes a knowledge of the extent and characteristics of trucking highly desirable and such knowledge is a prerequisite to a thorough understanding of our national transportation problems. The obtaining of data which would supply such knowledge was one of the objectives of the highway planning surveys, conducted by the various States in cooperation with the Public Roads Administration, beginning in 1936. The measuring of road mileages, traffic counts by vehicle type, the weighing of trucks, the questioning of truck drivers concerning origin and destination, and the questioning of truck owners concerning the miles driven on different road systems during the preceding year, supplied the basic data on which estimates of vehicle-miles driven and ton-miles hauled on the different road systems, and between origins and destinations in different categories, could be based.

These highway planning survey operations were extensive. Measurements were made of the mileage of all rural roads open to public use and scheduled traffic counts were made during a 1-year period at times and places sufficient to permit fairly accurate estimates of the volume of traffic on all portions of this mileage. The road-use survey, in which representative owners were questioned concerning miles driven during the preceding year, afforded a check of the vehicle-miles calculated from the traffic survey and supplied this information for a few States that had not completed traffic tables. In the loadometer survey, more than 2½ million trucks were weighed. This operation was carried out principally on main roads, as the number of trucks passing a given point on a local road in the course of a day was generally so small that the results to be obtained would not justify the cost. Origin and destination information was obtained at all loadometer stations, and, in many States, at a number of additional stations on local roads.

In spite of the large volume of pertinent data available in the highway planning survey tables, there are still deficiencies in information needed to compute the ton-miles of load hauled in a specific year on main and local rural roads throughout the United States. For example, a few States have not compiled traffic tables, and several have not compiled loadometer tables. As pointed out in the preceding paragraph, there is a scarcity of data concerning weights of vehicles on local roads.

Since field surveys were made in different years in different States it was necessary to establish trends from the survey year to a common year. Continuing survey operations give ample data for estimating trends

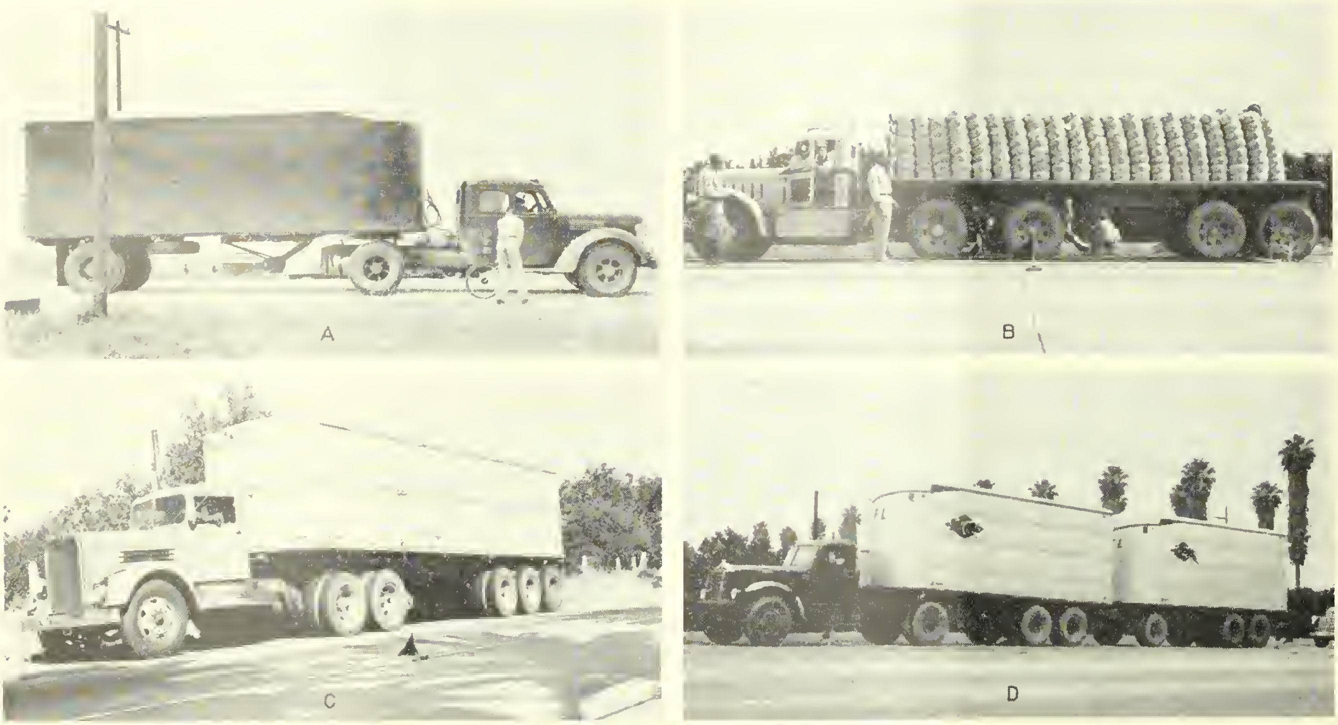

Figure 1.—Several Kinds of Truck Combinations.

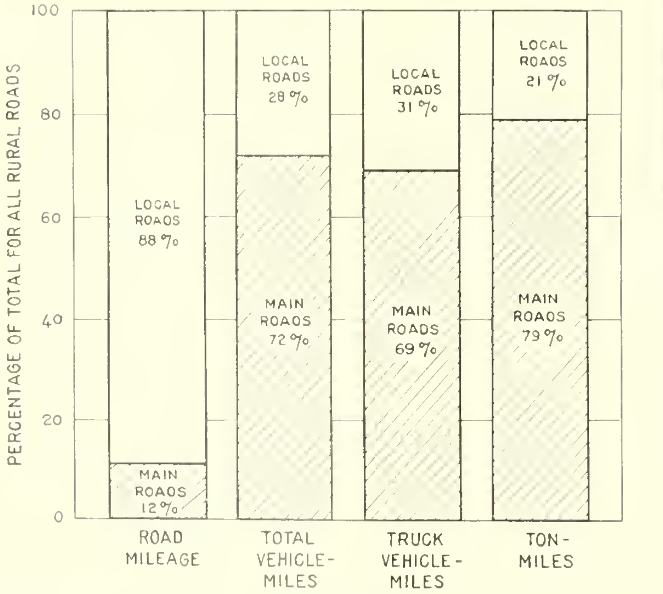

Figure 2.—Comparison of Road Mileage, Total Vehicle-Miles, Truck Vehicle-Miles and Ton-Miles of Carried Load on Main Rural Roads With Those on Local Rural Roads, in the Year 1940.

in vehicle-miles, but weighing operations were repeated in only a few States in the 1936–40 period, and the basis for estimating weight trends is therefore not entirely satisfactory. Fortunately, local roads carry only a small percentage of the total ton-mileage, and weight changes during the period considered were not large, so that errors of considerable proportions in estimating local-road tonnages and weight trends would introduce only small percentage errors in the over-all totals.

On the whole, it was thought that the available data were sufficient to permit the preparation of estimates which, though not entirely accurate in some details, would nevertheless give a comprehensive picture of the amount and characteristics of trucking on rural roads which would have considerable reliability in its broad outlines, and would lead to a better understanding of the function of highway transportation in the movement of commodities. Each figure used in the estimates was decided upon after all available data were carefully considered and an effort was made to find the reasons for any lack of agreement in data from two or more sources such as, for example, the traffic survey and the road-use survey. States lacking sufficient information to give a satisfactory basis for the estimates in a given table were lumped together in the table under the heading "Other States" though, in many cases, the estimates for the "Other States" group were arrived at by making the best estimate possible for each State in the group. The data on which each estimate is based, and the method of arriving at the figures will be described in detail in a later section of this article, after the estimates have been presented and discussed as to significance.

TRUCKING CHARACTERISTICS DIFFER ON MAIN AND LOCAL ROADS

The year 1940 was selected as the base for the estimates because it was the last year before war activities began to make important changes in traffic trends. Since trends were relatively stable between 1936 and 1940, estimates could be made with much greater assurance of accuracy for 1940 than for a later year. However, a special short survey made in the summer of 1942 afforded the basis for estimating wartime trends on main roads and estimates based on these data will be presented also.

Since trucking characteristics on local roads differ materially from those on main roads, it was thought advisable to prepare separate estimates for the two

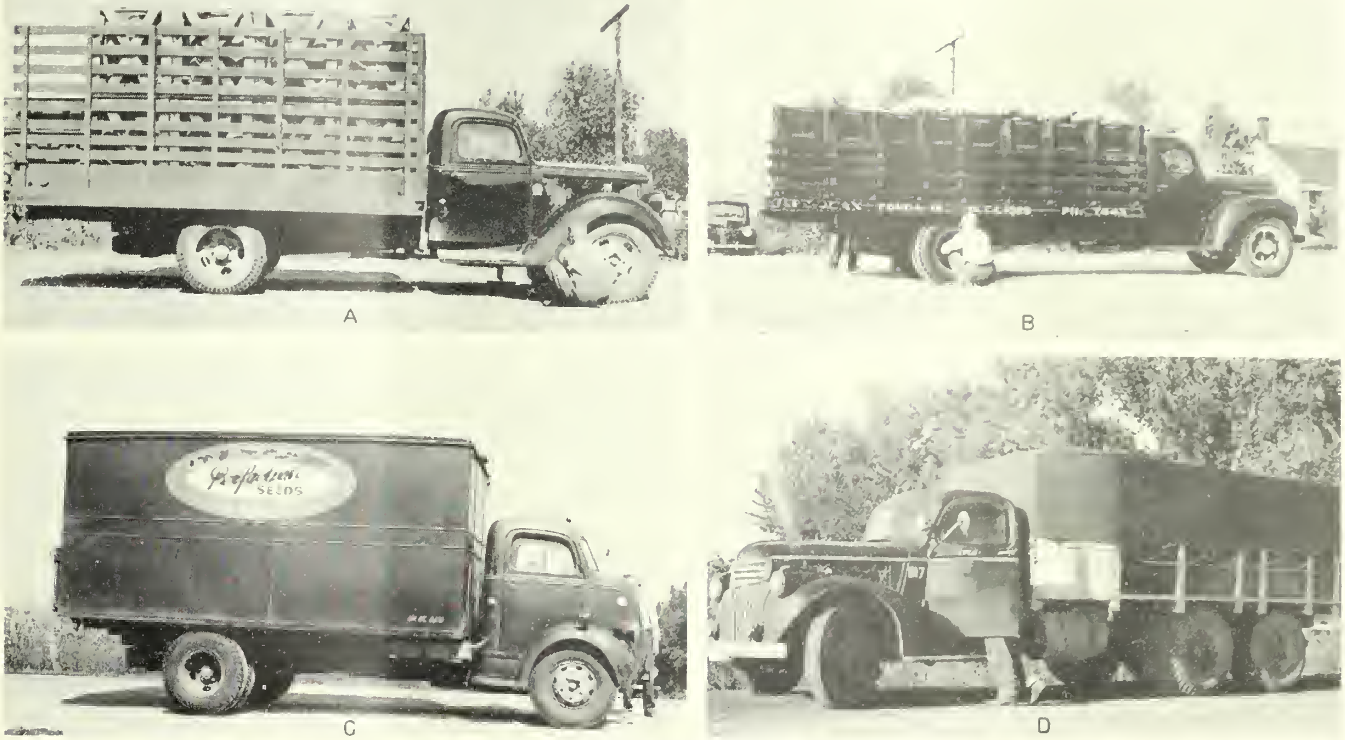

Figure 3.—Typical Single-Unit Trucks.

Table 1.—*Percentage of total rural road mileage, percentage of total rural vehicle-mileage of all vehicles, and average daily traffic density on main and on local rural roads in each State in the year 1940*

State	Main roads			Local roads			All roads—average daily vehicles
	Percentage of total mileage	Percentage of total vehicle-mileage	Average daily vehicles	Percentage of total mileage	Percentage of total vehicle-mileage	Average daily vehicles	
Alabama	10.4	63.2	625	89.6	36.8	42	102
Arizona	12.2	65.5	551	87.8	34.5	40	103
Arkansas	16.2	83.7	429	83.8	16.3	16	83
California	12.7	71.7	1,555	87.3	28.3	89	276
Colorado	15.8	83.7	373	84.2	16.3	14	70
Connecticut	21.2	84.4	1,885	78.8	15.6	94	473
Florida	22.4	76.8	779	77.6	23.2	68	227
Georgia	13.2	69.4	600	86.8	30.6	40	114
Idaho	13.6	73.2	478	86.4	26.8	27	89
Illinois	9.3	78.7	1,441	90.7	21.3	40	170
Indiana	11.5	70.0	1,206	88.5	30.0	67	198
Iowa	8.3	63.6	846	91.7	36.4	37	111
Kansas	7.1	62.7	621	92.9	37.3	28	70
Kentucky	15.4	78.7	631	84.6	21.3	31	124
Louisiana	9.1	70.2	1,028	90.9	29.8	44	134
Maryland	24.3	83.8	1,426	75.7	16.2	88	412
Massachusetts	10.1	55.7	3,532	89.9	44.3	316	641
Michigan	9.3	71.8	1,575	90.7	28.2	64	203
Minnesota	9.2	63.3	653	90.8	36.7	38	95
Mississippi	9.5	70.8	925	90.5	29.2	40	124
Missouri	11.8	80.5	981	88.2	19.5	20	91
Montana	7.7	64.5	370	92.3	35.5	18	47
Nebraska	8.0	64.3	436	92.0	35.7	21	54
Nevada	11.5	79.4	311	88.5	20.6	11	45
New Hamsphire	10.9	69.2	1,367	89.1	30.8	74	214
New Mexico	5.8	65.7	530	94.2	34.3	17	47
North Carolina	18.0	75.1	636	82.0	24.9	54	176
North Dakota	6.4	67.8	183	93.6	32.2	6	17
Ohio	17.7	76.5	1,089	82.3	23.5	65	251
Oklahoma	8.1	71.6	795	91.9	28.4	28	89
Oregon	13.6	73.2	597	86.4	26.8	34	111
Pennsylvania	14.7	73.0	1,347	85.3	27.0	86	272
Rhode Island	29.5	85.0	1,697	70.5	15.0	125	589
South Carolina	19.8	80.1	574	80.2	19.9	35	142
South Dakota	6.0	59.5	317	94.0	40.5	14	32

Table 1.—*Percentage of total rural road mileage, percentage of total rural vehicle-mileage of all vehicles, and average daily traffic density on main and on local rural roads in each State in the year 1940*—Continued

State	Main roads			Local roads			All roads—average daily vehicles
	Percentage of total mileage	Percentage of total vehicle-mileage	Average daily vehicles	Percentage of total mileage	Percentage of total vehicle-mileage	Average daily vehicles mileage	
Tennessee	10.8	71.6	770	89.2	28.4	37	116
Texas	11.2	70.8	921	88.8	29.2	48	146
Utah	13.0	75.0	847	87.0	25.0	30	102
Vermont	12.6	76.6	776	87.4	23.4	35	128
Virginia	19.8	78.6	889	80.2	21.4	60	224
Washington	12.1	72.7	996	87.9	27.3	41	137
West Virginia	13.6	68.0	869	86.4	32.0	65	174
Wisconsin	11.2	65.8	838	88.8	34.2	55	142
Wyoming	15.0	80.1	448	85.0	19.9	20	84
Subtotal	11.4	69.0	841	88.6	31.0	42	133
Other States [1]	15.5	71.9	1,545	84.5	28.1	127	347
Totals and averages	11.6	71.6	883	88.4	28.4	46	143

[1] Includes Delaware, Maine, New Jersey, and New York

classes of roads. Another reason for this decision was that the data for main roads were much more extensive than those for local roads and the separate treatment of the two systems permitted the segregation of the estimates well supported by data from those which were more speculative.

There is considerable difference in the manner of classifying roads into administrative systems in the different States, but in general the State system, or the primary State system in some States, comprises the mileage classed as "main." The term "local roads," as used in this analysis, includes important county roads and secondary State roads, as well as the roads which render little service except to the people living along them.

Figure 2 and tables 1 and 2 give comparisons of mileages and traffic service rendered by main and local roads and tend to define the classifications, as used in this discussion. The main roads constitute only a small percentage of the rural road mileage, but accommodate a high percentage of the vehicle-mileage of both passenger cars and trucks, and a still higher percentage of the ton-mileage of commodities hauled. Average daily traffic density was nearly twenty times as great on main roads as on local roads. It can be seen, then, that in spite of some indefiniteness at the borderline, the two classifications represent on the whole, distinct classes of roads with very different characteristics as regards traffic service.

TON-MILEAGE ON MAIN ROADS DIVIDED ALMOST EQUALLY BETWEEN SINGLE-UNIT TRUCKS AND COMBINATIONS

A considerable portion of the truck traffic on main roads consisted of combinations like those shown in figure 1. Table 3 shows that, for the country as a whole, 21.0 percent of the truck vehicle-mileage on main roads was by these combinations. The percentage varied greatly in different States, however. The highest figure was 40.0 percent in Indiana and the lowest was 5.1 percent in nearby Kentucky. The reason for the small number of combinations in Kentucky was a State law which limited gross weights to 18,000 pounds and made uneconomical the use of any type of vehicle larger than a single-unit truck. This law has not yet been changed permanently, though the restriction has been relaxed to some extent during the war, on roads designated as "National Emergency Highways."

On local roads, the major portion of the truck traffic was by single-unit trucks such as are shown in figure 3, and only about 6 percent was by combinations (table 4). In several States, however, the combinations amounted to from 10 to 17 percent of the truck traffic on roads of this class.

The proportion of loaded vehicles was greater for combinations than for single-unit trucks. Table 5 shows that for all rural roads, both main and local,

TABLE 2.—*Vehicle-mileage of all vehicles and of trucks and combinations on main and on local rural roads in each State in the year 1940*

State	Main roads			Local roads			All roads		
	All vehicles	Trucks and combinations		All vehicles	Trucks and combinations		All vehicles	Trucks and combinations	
	Thousands of vehicle-miles	*Percent*	*Thousands of vehicle-miles*	*Thousands of vehicle-miles*	*Percent*	*Thousands of vehicle-miles*	*Thousands of vehicle-miles*	*Percent*	*Thousands of vehicle-miles*
Alabama	1, 393, 588	21. 4	298, 228	811, 456	34. 2	277, 518	2, 205, 044	26. 1	575, 746
Arizona	675, 909	17. 1	115, 580	356, 013	23. 7	84, 375	1, 031, 922	19. 4	199, 955
Arkansas	1, 376, 409	28. 1	386, 771	268, 046	34. 2	91, 672	1, 644, 455	29. 1	478, 443
California	7, 171, 423	15. 2	1, 090, 056	2, 830, 561	23. 1	653, 860	10, 001, 984	17. 4	1, 743, 916
Colorado	1, 617, 112	22. 6	365, 467	314, 921	27. 8	87, 548	1, 932, 033	23. 4	453, 015
Connecticut	1. 685, 135	15. 5	261, 196	311, 470	21. 5	66, 966	1, 996, 605	16. 4	328, 162
Florida	1, 843, 854	24. 8	457, 276	556, 997	31. 2	173, 783	2, 400, 851	26. 3	631, 059
Georgia	2, 816, 554	21. 5	605, 559	1, 241, 881	25. 0	310, 470	4, 058, 435	22. 6	916, 029
Idaho	797, 605	21. 1	168, 295	292, 019	31. 7	92, 570	1, 089, 624	23. 9	260, 865
Illinois	5, 019, 360	16. 9	848, 272	1, 358, 480	25. 9	351, 846	6, 377, 840	18. 8	1, 200, 118
Indiana	3, 795, 926	21. 0	797, 144	1, 626, 825	24. 0	390, 438	5, 422, 751	21. 9	1, 187, 582
Iowa	2, 607, 397	18. 0	469, 331	1, 492, 284	16. 0	238, 765	4, 099, 681	17. 3	708, 096
Kansas	2, 060, 368	19. 7	405, 892	1, 225, 706	22. 5	275, 784	3, 286, 074	20. 7	681, 676
Kentucky	2, 001, 003	20. 4	408, 204	541, 568	25. 8	139, 725	2, 542, 571	21. 6	547, 929
Louisiana	1, 306, 449	21. 9	286, 112	554, 590	29. 1	161, 386	1, 861, 039	24. 0	447, 498
Maryland	2, 016, 661	21. 3	429, 549	389, 856	18. 6	72, 513	2, 406, 517	20. 9	502, 062
Massachusetts	2, 267, 270	15. 0	340, 091	1, 803, 233	18. 0	324, 582	4, 070, 503	16. 3	664, 673
Michigan	4, 739, 382	16. 1	763, 041	1, 861, 429	17. 5	325, 750	6, 600, 811	16. 5	1, 088, 791
Minnesota	2, 383, 904	18. 2	433, 872	1, 382, 137	16. 8	232, 199	3, 766, 041	17. 7	666, 071
Mississippi	1, 942, 736	27. 4	532, 310	801, 241	30. 0	240, 372	2, 743, 977	28. 1	772, 682
Missouri	3, 111, 217	22. 3	693, 801	753, 649	23. 9	180, 122	3, 864, 866	22. 6	873, 923
Montana	864, 395	21. 1	182, 387	475, 753	24. 7	117, 511	1, 340, 148	22. 4	299, 898
Nebraska	1, 270, 841	20. 0	254, 168	705, 583	22. 2	156, 639	1, 976, 424	20. 8	410, 807
Nevada	304, 573	16. 2	49, 341	79, 020	33. 1	26, 156	383, 593	19. 7	75, 497
New Hampshire	677, 918	14. 4	97, 620	301, 733	16. 2	48, 881	979. 651	15. 0	116, 501
New Mexico	686, 210	23. 8	163, 318	358, 249	31. 5	112, 848	1, 044, 459	26. 4	276, 166
North Carolina	2, 738, 509	24. 7	676, 412	907, 974	17. 9	162, 527	3, 646, 483	23. 0	838, 939
North Dakota	470, 539	20. 4	95, 990	223, 472	20. 6	46, 035	694, 011	20. 5	142, 025
Ohio	5. 783, 870	17. 1	989, 042	1, 776, 745	21. 2	376, 670	7, 560, 615	18. 1	1, 365, 712
Oklahoma	2, 378, 667	20. 6	490, 005	943, 493	21. 7	204, 738	3, 322, 160	20. 9	694, 743
Oregon	1, 386, 134	17. 0	235, 643	507, 491	18. 5	93, 886	1, 893, 625	17. 4	329, 529
Pennsylvania	6, 143, 806	17. 3	1, 062, 878	2, 272, 366	15. 8	359, 034	8, 416, 172	16. 9	1, 421, 912
Rhode Island	424. 127	12. 5	53, 016	74, 846	15. 0	11, 227	498, 973	12. 9	64, 243
South Carolina	1, 850, 967	20. 2	373, 895	459, 853	23. 5	108, 065	2, 310, 820	20. 8	481, 960
South Dakota	700, 406	20. 2	141, 482	476, 747	22. 2	105, 838	1, 177, 153	21. 0	247, 320
Tennessee	1, 903, 731	20. 1	382, 650	755, 111	30. 4	229, 554	2, 658, 842	23. 0	612, 204
Texas	6, 982, 043	20. 4	1, 424, 337	2, 879, 600	19. 3	555, 763	9, 861, 643	20. 1	1, 980, 100
Utah	601, 706	18. 4	110, 714	200, 568	24. 4	48, 939	802, 274	19. 9	159, 653
Vermont	481, 600	13. 7	65, 979	147, 121	28. 8	42, 371	628, 721	17. 2	108, 350
Virginia	2, 960, 243	19. 7	583, 168	805, 970	21. 0	169, 254	3, 766, 213	20. 0	752, 422
Washington	2, 218, 585	15. 7	348, 318	833, 114	17. 4	144, 962	3, 051, 699	16. 2	493, 280
West Virginia	1, 405, 272	17. 6	247, 328	661, 305	21. 7	143, 503	2, 066, 577	18. 9	390, 831
Wisconsin	2, 816, 531	17. 5	492, 893	1, 463, 911	20. 8	304, 493	4, 280, 442	18. 6	797, 386
Wyoming	604, 242	17. 5	105, 742	150, 118	29. 9	44, 885	754, 360	20. 0	150, 627
Subtotal	98, 284, 177	19. 1	18, 782, 373	38, 234, 505	21. 9	8, 386, 023	136, 518, 682	19. 9	27, 168, 396
Other States [1]	11, 345, 443	15. 7	1, 775, 933	5, 081, 169	17. 8	905, 972	16, 426, 612	16. 3	2, 681, 905
Totals and averages	109, 629, 620	18. 8	20, 558, 306	43, 315, 674	21. 5	9, 291, 995	152, 945, 294	19. 5	29, 850, 301

[1] Includes Delaware, Maine, New Jersey, and New York.

about 72 percent of the combinations compared to about 64 percent of the single-unit trucks were loaded. Figure 4 shows separately the vehicle-mileage of loaded and empty single-unit trucks and combinations on main and local rural roads for the entire year 1940.

Figure 5 was plotted from data in tables 3 and 4. The vehicle-mileage of loaded vehicles is represented by a horizontal measurement and the average carried load is represented by a vertical measurement so that ton-mileage, the product of the two, is represented by a rectangular area. The ton-mileage of load carried on main roads was divided about equally between single-unit trucks and combinations, in spite of the fact that 79 percent of the vehicles in the traffic stream were single-unit trucks. This is the result of the heavier loads carried by the combinations, averaging 7.58 tons compared to 2.24 tons for single-unit trucks. The average carried loads of combinations were much heavier in the western States than in other sections of the country, being over 10 tons in California, Nevada, and Washington, and, at the other extreme, under 5 tons in Arkansas, Kentucky, South Dakota, and Texas. The reason for the high average carried load in the western States was the prevalence of vehicles like those

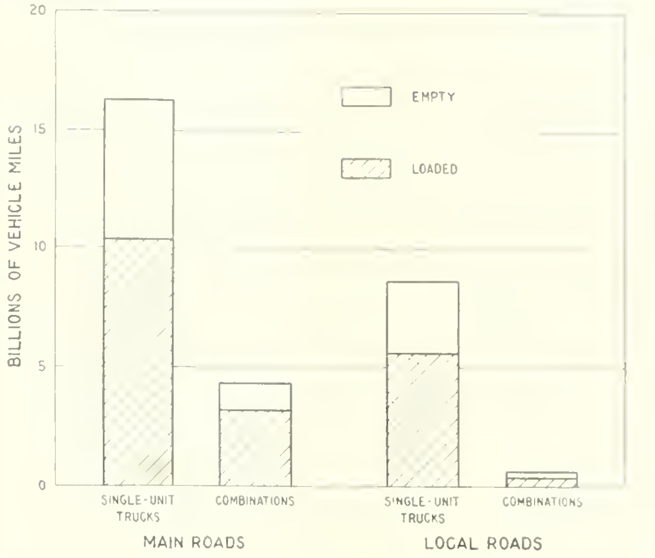

Figure 4.—Vehicle-Miles of Loaded and Empty Single-Unit Trucks and Combinations on Main and Local Rural Roads, in the Year 1940.

Table 3.—*Vehicle-mileage of trucks and combinations, percentage loaded, average carried load, and ton-mileage of carried load, on MAIN RURAL ROADS in each State in the year 1940*

State	Single-unit trucks					Truck combinations					All trucks and combinations			
	Percentage of all trucks	Vehicle-miles	Percentage loaded	Carried load		Percentage of all trucks	Vehicle-miles	Percentage loaded	Carried load		Vehicle-miles	Percentage loaded	Carried load	
				Average weight	Ton-miles				Average weight	Ton-miles			Average weight	Ton-miles
		Thousands		*Tons*	*Thousands*		*Thousands*		*Tons*	*Thousands*	*Thousands*		*Tons*	*Thousands*
Alabama	83.3	248,424	58.2	2.20	318,083	16.7	49,804	63.6	6.15	194,801	298,228	59.1	2.91	512,884
Arizona	79.8	92,233	59.9	2.79	154,142	20.2	23,347	72.1	9.89	166,478	115,580	62.3	4.45	320,620
Arkansas	83.2	321,793	59.0	1.64	311,367	16.8	64,978	69.5	4.84	218,574	386,771	60.8	2.25	529,941
California	72.7	792,471	69.1	2.22	1,215,665	27.3	297,585	80.5	12.63	3,025,592	1,090,056	72.2	5.39	4,241,257
Colorado	92.3	337,326	68.4	2.42	558,369	7.7	28,111	71.1	6.69	133,854	365,467	68.6	2.76	692,223
Florida	83.2	380,454	47.1	2.77	496,367	16.8	76,822	62.0	7.83	372,943	457,276	49.6	3.83	869,310
Georgia	76.1	460,830	68.4	2.45	772,260	23.9	144,729	70.3	7.30	742,731	605,559	68.9	3.63	1,514,991
Idaho	92.3	155,336	59.7	1.82	168,780	7.7	12,959	78.1	5.50	55,666	168,295	61.1	2.18	224,446
Illinois	72.5	614,997	63.7	2.35	920,620	27.5	233,275	71.7	7.43	1,242,727	848,272	65.9	3.87	2,163,347
Indiana	60.0	478,286	66.3	1.95	618,353	40.0	318,858	78.6	6.93	1,737,810	797,144	71.2	4.15	2,356,163
Iowa	81.2	381,097	70.1	2.04	544,984	18.8	88,234	74.1	7.38	482,512	469,331	70.9	3.09	1,027,496
Kansas	66.2	268,701	61.2	1.97	323,957	33.8	137,191	63.5	6.80	592,389	405,892	62.0	3.64	916,346
Kentucky	94.9	387,386	68.0	2.76	727,045	5.1	20,818	75.8	4.68	73,850	408,204	68.4	2.87	800,895
Louisiana	78.9	225,742	58.5	2.17	286,568	21.1	60,370	60.7	5.12	187,622	286,112	59.0	2.81	474,190
Maryland	80.4	345,357	67.3	2.57	597,332	19.6	84,192	74.3	8.00	500,440	429,549	68.7	3.72	1,097,772
Massachusetts	88.9	302,341	72.8	2.65	583,276	11.1	37,750	79.7	9.30	279,809	340,091	73.6	3.45	863,085
Michigan	71.1	542,522	69.1	1.78	667,292	28.9	220,519	68.9	7.46	1,133,457	763,041	69.0	3.42	1,800,749
Minnesota	85.2	369,659	68.3	2.22	560,499	14.8	64,213	79.4	6.37	324,774	433,872	69.9	2.92	885,273
Mississippi	88.6	471,627	60.6	1.79	511,593	11.4	60,683	65.9	6.18	247,138	532,310	61.2	2.33	758,731
Missouri	83.4	578,630	60.6	2.42	848,573	16.6	115,171	72.3	7.12	592,875	693,801	62.5	3.32	1,441,448
Montana	87.9	160,318	50.0	2.81	225,247	12.1	22,069	63.5	7.96	111,551	182,387	51.6	3.58	336,798
Nebraska	85.9	218,330	68.4	2.23	333,024	14.1	35,838	70.2	6.22	156,483	254,168	68.7	2.80	489,507
Nevada	87.6	43,223	62.0	2.62	70,211	12.4	6,118	78.8	10.21	49,222	49,341	64.1	3.78	119,433
New Hampshire	90.8	88,639	73.6	2.37	154,614	9.2	8,981	77.1	6.33	43,829	97,620	73.9	2.75	198,443
New Mexico	89.4	146,006	60.2	2.47	217,103	10.6	17,312	68.8	6.31	75,516	163,318	61.1	2.93	292,619
North Carolina	72.8	492,428	63.7	2.17	680,679	27.2	183,984	71.8	8.04	1,062,092	676,412	65.9	3.91	1,742,771
North Dakota	87.9	84,375	65.5	2.79	154,192	12.1	11,615	81.4	5.75	54,366	95,990	67.4	3.22	208,558
Ohio	66.0	652,768	57.5	3.01	1,129,779	34.0	336,274	72.7	8.53	2,085,338	989,042	62.7	5.19	3,215,117
Oklahoma	73.8	361,624	55.8	1.96	395,501	26.2	128,381	70.6	6.29	570,107	490,005	59.7	3.30	965,608
Pennsylvania	86.8	922,578	66.6	2.35	1,443,927	13.2	140,300	69.5	8.08	787,873	1,062,878	66.7	3.13	2,231,800
Rhode Island	92.2	48,881	65.1	2.30	73,191	7.8	4,135	73.2	6.72	20,341	53,016	65.7	2.68	93,532
South Carolina	64.8	242,284	66.3	2.19	351,788	35.2	131,611	65.4	7.16	616,290	373,895	66.0	3.92	968,078
South Dakota	91.3	129,173	63.6	2.01	165,130	8.7	12,309	75.1	4.71	43,539	141,482	64.6	2.28	208,669
Tennessee	91.8	351,273	63.7	2.41	539,264	8.2	31,377	77.0	5.85	141,336	382,650	64.8	2.75	680,600
Texas	80.0	1,139,470	59.6	1.76	1,195,258	20.0	284,867	66.0	4.25	799,051	1,424,337	60.9	2.30	1,994,309
Utah	90.0	99,643	63.5	2.35	148,692	10.0	11,071	70.3	7.77	60,474	110,714	64.2	2.94	209,166
Virginia	82.1	478,781	65.7	2.43	764,378	17.9	104,387	72.2	7.99	602,182	583,168	66.3	3.53	1,366,560
Washington	85.5	297,812	59.7	2.31	410,704	14.5	50,506	68.7	10.41	361,206	348,318	61.0	3.63	771,910
West Virginia	94.1	232,736	60.4	2.34	328,941	5.9	14,592	71.5	6.75	70,423	247,328	61.1	2.64	399,364
Wisconsin	75.5	372,134	72.7	1.68	454,509	24.5	120,759	81.1	7.00	685,552	492,893	74.7	3.10	1,140,061
Wyoming	77.3	81,739	56.0	3.26	149,223	22.7	24,003	64.7	6.64	103,119	105,742	58.0	4.12	252,342
Subtotal	79.0	14,399,427	63.7	2.24	20,570,480	21.0	3,820,128	71.8	7.58	20,805,932	18,219,555	65.4	3.47	41,376,412
Other States [1]	79.0	1,847,613	63.7	2.24	2,636,321	21.0	491,138	71.8	7.58	2,672,988	2,338,751	65.4	3.47	5,309,309
Totals and averages	79.0	16,247,040	63.7	2.24	23,206,801	21.0	4,311,266	71.8	7.58	23,478,920	20,558,306	65.4	3.47	46,685,721

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[1] Includes Connecticut, Delaware, Maine, New Jersey, New York, Oregon, and Vermont.

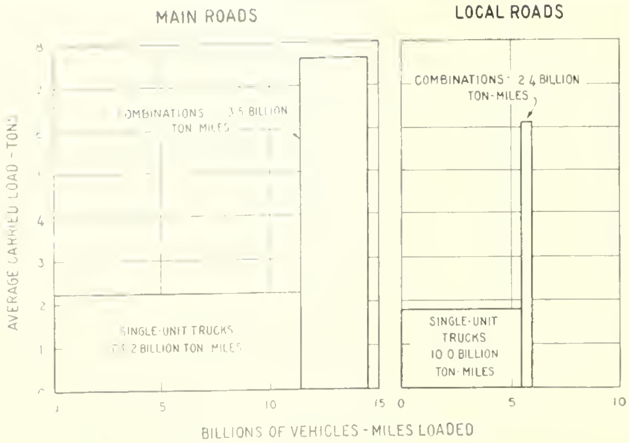

FIGURE 5.—TON-MILES OF LOAD CARRIED BY SINGLE-UNIT TRUCKS AND COMBINATIONS ON MAIN AND LOCAL RURAL ROADS, IN THE YEAR 1940.

shown in figures 1B, 1C, and 1D. Because of State laws, and for other reasons, the use of these large vehicles is confined mainly to the western States, while 3-axle combinations of the type shown in figure 1A are widely used throughout the country.

Figure 5 shows that, on local roads, about one-fifth of the total tonnage was hauled by combinations. This is a surprisingly high portion, when it is considered that less than 1 truck in 15 found on these roads was a combination.

ORIGINS AND DESTINATIONS CLASSIFIED AS RURAL AND URBAN

One of the questions in which students of the economics of transportation are especially interested is the extent to which the movement of freight by highway is competitive with movement by rail or water, and the extent to which it is tributary, or supplemental to other transport. This question could be clearly answered only after a thorough study of origins and destinations of individual trucks passing specific stations, as recorded in the survey notes. It would be necessary to consider the possibility of alternate means of transport in each individual case in order to evaluate accurately the competitive movement. To make such a detailed study on a Nation-wide, or even a State-wide

TABLE 4.—*Vehicle-mileage of trucks and combinations, percentage loaded, average carried load, and ton-mileage of carried load, on LOCAL RURAL ROADS in each State in the year 1940*

State	Single-unit trucks					Truck combinations					All trucks and combinations			
				Carried load					Carried load				Carried load	
	Percentage of all trucks	Vehicle-miles	Percentage loaded	Average weight	Ton-miles	Percentage of all trucks	Vehicle-miles	Percentage loaded	Average weight	Ton-miles	Vehicle-miles	Percentage loaded	Average weight	Ton-miles
		Thousands		*Tons*	*Thousands*		*Thousands*		*Tons*	*Thousands*	*Thousands*		*Tons*	*Thousands*
Alabama	86.9	241,163	58.2	1.76	247,028	13.1	36,355	63.6	4.92	113,760	277,518	58.9	2.21	360,788
Arizona	95.0	80,156	59.9	2.23	107,069	5.0	4,219	72.1	7.91	24,062	84,375	60.5	2.57	131,131
Arkansas	83.2	76,271	59.0	1.31	58,950	16.8	15,401	69.5	3.88	41,532	91,672	60.8	1.80	100,482
California	91.7	599,590	69.1	1.77	733,341	8.3	54,270	80.5	10.10	441,239	653,860	70.6	2.56	1,174,580
Colorado	97.3	85,184	68.4	1.94	113,036	2.7	2,364	71.1	5.35	8,993	87,548	68.5	2.04	122,029
Florida	95.1	165,268	47.1	2.22	172,807	4.9	8,515	62.0	6.26	33,047	173,783	47.8	2.48	205,854
Georgia	92.8	288,116	68.4	1.96	386,259	7.2	22,354	70.3	5.84	91,776	310,470	68.5	2.25	478,035
Idaho	94.1	87,108	59.7	1.46	75,924	5.9	5,462	78.1	4.40	18,770	92,570	60.8	1.68	94,694
Illinois	90.8	319,476	63.7	1.88	382,591	9.2	32,370	71.7	5.95	138,094	351,846	64.4	2.30	520,685
Indiana	96.2	375,601	66.3	1.56	388,476	3.8	14,837	78.6	5.54	64,607	390,438	66.8	1.73	453,083
Iowa	98.9	236,139	70.1	1.63	269,819	1.1	2,626	74.1	5.91	11,501	238,765	70.1	1.68	281,320
Kansas	96.2	265,304	61.2	1.57	254,915	3.8	10,480	63.5	5.44	36,203	275,784	61.3	1.72	291,118
Kentucky	99.1	138,467	68.0	2.21	208,089	0.9	1,258	75.8	3.74	3,568	139,725	68.1	2.23	211,657
Louisiana	89.8	144,925	58.5	1.73	146,671	10.2	16,461	60.7	4.10	40,967	161,386	58.7	1.98	187,638
Maryland	83.5	60,548	67.3	2.06	83,913	16.5	11,965	74.3	6.40	56,896	72,513	68.5	2.84	140,839
Massachusetts	96.7	313,871	72.8	2.12	484,416	3.3	10,711	79.7	7.44	63,515	324,582	73.0	2.31	547,931
Michigan	97.4	317,280	68.2	1.42	307,267	2.6	8,470	64.3	5.97	32,513	325,750	68.1	1.53	339,780
Minnesota	95.6	221,982	68.3	1.78	269,873	4.4	10,217	79.4	5.10	41,371	232,199	68.8	1.95	311,244
Mississippi	96.6	232,199	60.6	1.43	201,220	3.4	8,173	65.9	4.95	26,661	240,372	60.8	1.56	227,881
Missouri	95.3	171,656	60.0	1.93	198,778	4.7	8,466	72.3	5.70	34,890	180,122	60.6	2.14	233,668
Montana	96.4	113,281	50.0	2.25	127,442	3.6	4,230	63.5	6.37	17,110	117,511	50.5	2.44	144,552
Nebraska	99.4	155,699	68.4	1.79	190,631	.6	940	70.2	4.97	3,280	156,639	68.4	1.81	193,911
Nevada	98.7	25,816	62.0	2.10	33,613	1.3	340	78.8	8.17	2,190	26,156	62.2	2.20	35,803
New Hampshire	88.9	43,455	73.6	1.89	60,448	11.1	5,426	77.1	5.06	21,166	48,881	74.0	2.26	81,614
New Mexico	96.8	109,237	60.2	1.98	130,207	3.2	3,611	68.8	5.07	12,594	112,848	60.5	2.09	142,801
North Carolina	91.8	149,200	63.7	1.73	164,419	8.2	13,327	71.8	6.43	61,529	162,527	64.4	2.16	225,948
North Dakota	96.4	44,378	65.5	2.23	64,821	3.6	1,657	81.4	4.60	6,205	46,035	66.1	2.34	71,026
Ohio	89.8	338,250	57.5	2.41	468,731	10.2	38,420	72.7	6.83	190,769	376,670	59.1	2.97	659,500
Oklahoma	94.6	193,682	55.8	1.57	169,678	5.4	11,056	70.8	5.03	39,375	204,738	56.6	1.80	209,053
Pennsylvania	95.2	341,800	66.6	1.88	427,961	4.8	17,234	69.5	6.46	77,378	359,034	66.7	2.11	505,339
Rhode Island	97.7	10,969	65.1	1.84	13,139	2.3	258	73.2	5.37	1,015	11,227	65.3	1.93	14,154
South Carolina	89.4	96,610	66.3	1.75	112,091	10.6	11,455	65.4	5.72	42,854	108,065	66.2	2.17	154,945
South Dakota	91.3	96,630	63.6	1.60	98,331	8.7	9,208	75.1	3.77	26,070	105,838	64.6	1.82	124,401
Tennessee	99.0	227,258	63.7	1.93	279,393	1.0	2,296	77.0	4.68	8,274	229,554	63.8	1.96	287,667
Texas	94.0	522,417	59.6	1.41	439,019	6.0	33,346	66.0	3.40	74,827	555,763	60.0	1.54	513,846
Utah	97.0	47,471	63.5	1.88	56,671	3.0	1,468	70.3	6.22	6,419	48,939	63.7	2.02	63,090
Virginia	94.8	160,453	65.7	1.95	205,565	5.2	8,801	72.2	6.39	40,602	169,254	66.0	2.20	246,167
Washington	94.5	136,989	59.7	1.85	151,297	5.5	7,973	68.7	8.33	45,623	144,962	60.1	2.26	196,920
West Virginia	99.2	142,355	60.4	1.88	161,646	.8	1,148	71.5	5.40	4,433	143,503	60.5	1.91	166,079
Wisconsin	92.6	281,961	72.7	1.34	274,681	7.4	22,532	81.1	5.60	102,329	304,493	73.3	1.69	377,010
Wyoming	97.2	43,628	56.0	2.61	63,768	2.8	1,257	64.7	5.31	4,317	44,885	56.2	2.70	68,085
Subtotal	94.1	7,701,843	64.0	1.78	8,784,024	5.9	480,957	71.6	6.13	2,112,324	8,182,800	64.4	2.07	10,896,348
Other States[1]	94.1	1,043,752	64.0	1.78	1,189,042	5.9	65,443	71.6	6.13	287,233	1,109,195	64.4	2.07	1,476,275
Totals and averages	94.1	8,745,595	64.0	1.78	9,973,066	5.9	546,400	71.6	6.13	2,399,557	9,291,995	64.4	2.07	12,372,623

[1] Includes Connecticut, Delaware, Maine, New Jersey, New York, Oregon, and Vermont.

basis, would be a vast undertaking, at a cost entirely out of proportion to the results achieved. However, a study of the estimates based on the general origin and destination tables of the highway planning survey will permit the placing of probable minimum and maximum limits on the extent of the competitive movement and will lead, in other respects, to a clearer understanding of the function of trucks, and the highways on which they travel, in the movement of goods.

In tables 6 to 11, inclusive, the origins and destinations are classified as urban or rural. Urban areas are defined as compactly built-up places with more than 1,000 inhabitants, whether incorporated or not, and all other places are classed as rural. In some sections of the country a town of eight or nine hundred inhabitants may be of considerable importance but it is nevertheless classed as rural under this definition.

A large part of the movement from rural origin to rural destination is undoubtedly between the farm and the nearest small town or railroad siding, and is therefore tributary or supplemental rather than competitive. It is possible that some freight travels by highway from a rural area to a distant small town or rural area, but it is improbable that such movements are of large proportions.

The movement between cities with more than 1,000 population, classed as "urban-urban," is more largely competitive, though it is probable that some of this movement is induced by the availability of truck service and would not otherwise take place. There are special cases where cities are separated by only a very narrow rural area and the movement is practically intracity, but because of the short distance traveled on rural roads, the vehicle-mileage or ton-mileage is small compared to the traffic volume or tonnage hauled. If the boundaries of two cities are actually contiguous, the movement does not enter into the tables since there is no driving on rural roads.

The movement from rural origin to urban destination is, to a considerable extent, a short-haul movement from the farm to a market or a rail or water shipping point, but the classification also includes some long-haul

TABLE 5.—*Vehicle-mileage of trucks and combinations, percentage loaded, average carried load, and ton-mileage of carried load on ALL RURAL ROADS in each State in the year 1940*

State	Single-unit trucks					Truck combinations					All trucks and combinations			
				Carried load					Carried load				Carried load	
	Percentage of all trucks	Vehicle-miles	Percentage loaded	Average weight	Ton-miles	Percentage of all trucks	Vehicle-miles	Percentage loaded	Average weight	Ton-miles	Vehicle-miles	Percentage loaded	Average weight	Ton-miles
		Thousands		*Tons*	*Thousands*		*Thousands*		*Tons*	*Thousands*	*Thousands*		*Tons*	*Thousands*
Alabama	85.0	489, 587	58.2	1.98	565, 111	15.0	86, 159	63.6	5.63	308, 561	575, 746	59.0	2.57	873, 672
Arizona	86.2	172, 389	59.9	2.53	261, 211	13.8	27, 566	72.1	9.59	190, 540	199, 955	61.6	3.67	451, 751
Arkansas	83.2	398, 064	59.0	1.58	370, 317	16.8	80, 379	69.5	4.66	260, 106	478, 443	60.8	2.17	630, 423
California	79.8	1, 392, 061	69.0	2.03	1, 919, 006	20.2	351, 855	80.5	12.24	3, 466, 831	1, 743, 916	71.4	4.35	5, 115, 837
Colorado	93.3	422, 510	68.4	2.32	671, 405	6.7	30, 505	71.1	6.59	142, 847	453, 015	68.6	2.62	814, 252
Florida	86.5	545, 722	47.1	2.60	669, 174	13.5	85, 337	62.0	7.67	405, 990	631, 059	49.1	3.47	1, 075, 164
Georgia	81.8	748, 946	68.4	2.26	1, 158, 519	18.2	167, 083	70.3	7.10	834, 507	916, 029	68.7	3.16	1, 993, 026
Idaho	92.9	242, 444	59.7	1.69	244, 701	7.1	18, 421	78.1	5.17	74, 436	260, 865	61.0	2.01	319, 140
Illinois	77.9	934, 473	63.7	2.19	1, 303, 211	22.1	265, 645	71.7	7.25	1, 380, 821	1, 200, 118	65.5	3.42	2, 684, 032
Indiana	71.9	853, 887	66.3	1.78	1, 006, 829	28.1	333, 695	78.6	6.87	1, 802, 417	1, 187, 582	69.8	3.39	2, 809, 246
Iowa	87.2	617, 236	70.1	1.88	814, 803	12.8	90, 860	74.1	7.34	494, 013	708, 096	70.6	2.62	1, 308, 816
Kansas	78.3	534, 005	61.2	1.77	578, 872	21.7	147, 671	63.5	6.70	628, 592	681, 676	61.7	2.87	1, 207, 464
Kentucky	96.0	525, 853	68.0	2.62	935, 134	4.0	22, 076	75.8	4.63	77, 418	547, 929	68.3	2.74	1, 012, 552
Louisiana	82.8	370, 667	58.5	2.00	433, 239	17.2	76, 831	60.7	4.90	228, 589	447, 498	58.9	2.51	661, 828
Maryland	80.8	405, 905	67.3	2.49	681, 275	19.2	96, 157	74.3	7.80	557, 336	502, 062	68.6	3.59	1, 238, 611
Massachusetts	92.7	616, 212	72.8	2.38	1, 067, 692	7.3	48, 461	79.7	8.89	343, 324	664, 673	73.3	2.90	1, 411, 016
Michigan	79.0	859, 802	68.8	1.65	974, 559	21.0	228, 989	68.7	7.41	1, 165, 970	1, 088, 791	68.8	2.86	2, 140, 529
Minnesota	88.8	591, 641	68.3	2.05	830, 372	11.2	74, 430	79.4	6.20	366, 115	666, 071	69.5	2.58	1, 195, 717
Mississippi	91.1	703, 826	60.6	1.67	712, 813	8.9	68, 856	65.9	6.03	273, 799	772, 682	61.1	2.09	986, 612
Missouri	85.9	750, 286	60.5	2.31	1, 047, 351	14.1	123, 637	72.3	7.02	627, 765	873, 923	62.1	3.08	1, 675, 116
Montana	91.2	273, 599	50.0	2.58	352, 689	8.8	26, 299	63.5	7.70	128, 661	299, 898	51.2	3.14	481, 350
Nebraska	91.0	374, 029	68.4	2.05	523, 655	9.0	36, 778	70.2	6.19	159, 763	410, 807	68.6	2.43	683, 418
Nevada	91.4	69, 039	62.0	2.43	103, 824	8.6	6, 458	78.8	10.10	51, 412	75, 497	63.4	3.21	155, 236
New Hampshire	90.2	132, 094	73.6	2.21	215, 062	9.8	14, 407	77.1	5.85	64, 995	146, 501	73.9	2.59	280, 057
New Mexico	92.4	255, 243	60.2	2.26	347, 310	7.6	20, 923	68.8	6.12	88, 110	276, 166	60.9	2.59	435, 420
North Carolina	76.5	641, 628	63.7	2.07	845, 098	23.5	197, 311	71.8	7.93	1, 123, 621	838, 939	65.6	3.58	1, 968, 719
North Dakota	90.7	128, 753	65.5	2.60	219, 013	9.3	13, 272	81.4	5.61	60, 571	142, 025	67.0	2.94	279, 584
Ohio	72.6	991, 018	57.5	2.81	1, 598, 510	27.4	374, 694	72.7	8.36	2, 276, 107	1, 365, 712	61.7	4.60	3, 874, 617
Oklahoma	79.9	555, 306	55.8	1.82	565, 179	20.1	139, 437	70.6	6.19	609, 482	694, 743	58.8	2.88	1, 174, 661
Pennsylvania	88.9	1, 264, 378	66.6	2.22	1, 871, 888	11.1	157, 534	69.5	7.90	865, 251	1, 421, 912	66.9	2.88	2, 737, 139
Rhode Island	93.2	59, 850	65.1	2.22	86, 330	6.8	4, 393	73.2	6.64	21, 356	64, 243	65.7	2.55	107, 686
South Carolina	70.3	338, 894	66.3	2.06	463, 879	29.7	143, 066	65.4	7.04	659, 141	481, 960	66.0	3.53	1, 123, 020
South Dakota	91.3	225, 803	63.6	1.83	263, 461	8.7	21, 517	75.1	4.31	69, 609	247, 320	64.6	2.08	333, 070
Tennessee	94.5	578, 531	63.7	2.22	818, 657	5.5	33, 673	77.0	5.77	149, 610	612, 204	61.4	2.45	968, 267
Texas	83.9	1, 661, 887	59.6	1.65	1, 634, 277	16.1	318, 213	66.0	4.16	873, 878	1, 980, 100	60.6	2.09	2, 508, 155
Utah	92.1	147, 114	63.5	2.20	205, 363	7.9	12, 539	70.3	7.59	66, 893	159, 653	64.0	2.66	272, 256
Virginia	85.0	639, 234	65.7	2.31	969, 943	15.0	113, 188	72.2	7.87	642, 784	752, 422	66.7	3.21	1, 612, 727
Washington	88.1	434, 801	59.7	2.17	562, 001	11.9	58, 479	68.7	10.43	406, 829	493, 280	60.8	3.23	968, 830
West Virginia	96.0	375, 091	60.4	2.17	490, 587	4.0	15, 740	71.5	6.65	74, 856	390, 831	60.8	2.38	565, 443
Wisconsin	82.0	654, 095	72.7	1.53	729, 190	18.0	143, 291	81.1	6.78	787, 881	797, 386	74.2	2.56	1, 517, 071
Wyoming	83.2	125, 367	56.0	3.03	212, 991	16.8	25, 260	64.7	6.57	107, 136	150, 627	57.5	3.70	320, 127
Subtotal	83.7	22, 101, 270	63.8	2.08	29, 354, 504	16.3	4, 301, 085	71.8	7.42	22, 918, 256	26, 402, 355	65.1	3.04	52, 272, 760
Other States [1]	83.9	2, 891, 365	63.8	2.07	3, 825, 363	16.1	556, 581	71.8	7.41	2, 960, 221	3, 447, 946	65.1	3.02	6, 785, 584
Totals and averages	83.7	24, 992, 635	63.8	2.08	33, 179, 867	16.3	4, 857, 666	71.8	7.42	25, 878, 477	29, 850, 301	65.1	3.04	59, 058, 344

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[1] Includes Connecticut, Delaware, Maine, New Jersey, New York, Oregon, and Vermont.

Table 6.—*Vehicle-mileage of trucks and combinations with both origin and destination rural, with one rural and the other urban, and with both urban on MAIN RURAL ROADS in each State in the year 1940*

State	Total truck travel	Origin and destination					
		Both rural		One rural—one urban		Both urban	
	Thousands of vehicle-miles	*Per cent*	*Thousands of vehicle-miles*	*Per cent*	*Thousands of vehicle-miles*	*Per cent*	*Thousands of vehicle-miles*
Alabama	298, 228	15. 5	46, 225	52. 4	156, 272	32. 1	95, 731
Arizona	115, 580	13. 5	15, 603	45. 8	52, 936	40. 7	47, 041
California	1, 090, 056	8. 0	87, 204	40. 3	439, 293	51. 7	563, 559
Colorado	365, 467	15. 7	57, 378	53. 0	193, 698	31. 3	114, 391
Florida	457, 276	14. 5	66, 305	42. 9	196, 171	12. 6	194, 800
Idaho	168, 295	22. 3	37, 530	52. 2	87, 850	25. 5	42, 915
Illinois	848, 272	6. 8	57, 683	33. 5	284, 171	59. 7	506, 418
Indiana	797, 144	10. 3	82, 106	35. 3	281, 392	54. 4	433, 646
Iowa	469, 331	11. 4	53, 504	50. 8	238, 420	37. 8	177, 407
Kansas	405, 892	9. 3	37, 748	42. 3	171, 692	48. 4	196, 452
Kentucky	408, 204	11. 7	47, 760	43. 3	176, 752	45. 0	183, 692
Louisiana	286, 112	11. 3	32, 330	41. 6	119, 023	47. 1	134, 759
Maryland	429, 519	8. 4	36, 082	40. 6	174, 397	51. 0	219, 070
Massachusetts	340, 091	5. 6	19, 045	16. 5	56, 115	77. 9	264, 931
Michigan	763, 041	7. 8	59, 517	27. 9	212, 889	64. 3	490, 635
Minnesota	433, 872	17. 6	76, 361	46. 4	201, 317	36. 0	156, 194
Mississippi	532, 310	14. 1	75, 056	49. 1	261, 364	36. 8	195, 890
Missouri	693, 801	21. 6	149, 861	48. 4	335, 800	30. 0	208, 140
Montana	182, 387	19. 1	31, 836	56. 8	103, 596	24. 1	43, 955
Nebraska	254, 168	15. 4	39, 142	44. 8	113, 867	39. 8	101, 159
Nevada	49, 341	19. 9	9, 819	53. 9	26, 595	26. 2	12, 927
New Hampshire	97, 620	13. 3	12, 983	39. 5	38, 560	47. 2	46, 077
New Mexico	163, 318	21. 0	34, 297	47. 5	77, 576	31. 5	51, 445
North Carolina	676, 412	8. 4	56, 819	34. 2	231, 333	57. 4	388, 260
North Dakota	95, 990	28. 3	27, 165	50. 3	48, 283	21. 4	20, 542
Ohio	989, 042	1. 5	14, 835	25. 3	250, 228	73. 2	723, 979
Oklahoma	490, 005	11. 6	56, 841	42. 6	208, 742	45. 8	224, 422
Oregon	235, 643	15. 5	36, 525	56. 2	132, 431	28. 3	66, 687
Pennsylvania	1, 062, 878	7. 6	80, 779	36. 1	383, 699	56. 3	598, 400
Rhode Island	53, 016	3. 9	2, 068	31. 8	16, 859	64. 3	34, 089
South Carolina	373, 895	9. 9	37, 016	41. 7	155, 914	48. 4	180, 965
South Dakota	141, 482	15. 0	21, 222	57. 1	80, 786	27. 9	39, 474
Tennessee	382, 650	12. 7	48, 597	45. 6	174, 488	41. 7	159, 565
Texas	1, 424, 337	11. 1	158, 101	46. 2	658, 044	42. 7	608, 192
Utah	110, 714	14. 6	16, 164	55. 9	61, 889	29. 5	32, 661
Virginia	583, 168	15. 6	90, 974	44. 0	256, 594	40. 4	235, 600
Washington	348, 318	10. 1	35, 180	51. 2	178, 339	38. 7	134, 799
West Virginia	247, 328	19. 4	47, 981	37. 8	93, 490	42. 8	105, 857
Wisconsin	492, 893	16. 1	79, 356	40. 1	197, 650	43. 8	215, 887
Wyoming	105, 742	11. 2	11, 843	47. 0	49, 699	41. 8	44, 200
Subtotal	17, 462, 868	11. 4	1, 989, 841	41. 1	7, 178, 214	47. 5	8, 294, 813
Other States [1]	3, 095, 138	9. 2	285, 145	36. 3	1, 122, 940	54. 5	1, 687, 353
Total and averages	20, 558, 306	11. 1	2, 274, 986	40. 4	8, 301, 154	48. 5	9, 982, 166

[1] Includes Arkansas, Connecticut, Delaware, Georgia, Maine, New Jersey, New York, and Vermont.

Table 7.—*Vehicle-mileage of trucks and combinations with both origin and destination rural, with one rural and the other urban, and with both urban on LOCAL RURAL ROADS in each State in the year 1940*

State	Total truck travel	Origin and destination					
		Both rural		One rural—one urban		Both urban	
	Thousands of vehicle-miles	*Per cent*	*Thousands of vehicle-miles*	*Per cent*	*Thousands of vehicle-miles*	*Per cent*	*Thousands of vehicle-miles*
Alabama	277, 518	50. 1	139, 037	43. 1	119, 610	6. 8	18, 871
Arizona	84, 375	50. 4	42, 525	43. 0	36, 281	6. 6	5, 569
California	653, 860	35. 2	230, 159	49. 6	324, 314	15. 2	99, 387
Colorado	87, 548	57. 9	50, 690	39. 7	34, 757	2. 4	2, 101
Florida	173, 783	43. 7	75, 943	45. 9	79, 766	10. 4	18, 074
Idaho	92, 570	51. 0	47, 211	45. 7	42, 304	3. 3	3, 055
Illinois	351, 846	50. 8	178, 738	42. 8	150, 590	6. 4	22, 518
Indiana	390, 438	48. 8	190, 534	44. 9	175, 307	6. 3	24, 597
Iowa	238, 765	46. 6	111, 265	47. 5	113, 413	5. 9	14, 087
Kansas	275, 784	52. 9	145, 890	41. 9	115, 553	5. 2	14, 341
Kentucky	139, 725	52. 9	73, 914	41. 9	58, 545	5. 2	7, 266
Louisiana	161, 386	49. 0	79, 079	43. 6	70, 364	7. 4	11, 943
Maryland	72, 513	44. 4	32, 196	41. 2	29, 875	14. 4	10, 442
Massachusetts	324, 582	25. 1	81, 470	56. 7	184, 038	18. 2	59, 074
Michigan	325, 750	25. 2	82, 089	49. 3	160, 595	25. 5	83, 066
Minnesota	232, 199	54. 8	127, 245	40. 6	94, 273	4. 6	10, 681
Mississippi	240, 372	47. 9	115, 138	44. 1	106, 004	8. 0	19, 230
Missouri	180, 122	56. 1	101, 048	40. 5	72, 950	3. 4	6, 124
Montana	117, 511	56. 8	66, 746	40. 2	47, 240	3. 0	3, 525
Nebraska	156, 639	57. 9	90, 694	36. 4	57, 017	5. 7	8, 928
Nevada	26, 156	55. 4	14, 490	40. 8	10, 672	3. 8	994
New Hampshire	48, 881	39. 6	19, 357	44. 7	21, 850	15. 7	7, 674
New Mexico	112, 848	61. 2	69, 063	36. 0	40, 625	2. 8	3, 160
North Carolina	162, 527	45. 5	73, 950	45. 1	73, 300	9. 4	15, 277
North Dakota	46, 035	60. 0	27, 621	38. 8	17, 862	1. 2	552
Ohio	376, 670	25. 1	94, 544	56. 7	213, 572	18. 2	68, 554
Oklahoma	204, 738	53. 6	109, 740	41. 6	85, 171	4. 8	9, 827
Oregon	93, 886	46. 6	43, 751	49. 8	46, 755	3. 6	3, 380
Pennsylvania	359, 034	36. 6	131, 406	49. 0	175, 927	14. 4	51, 701
Rhode Island	11, 227	25. 1	2, 818	56. 7	6, 366	18. 2	2, 043
South Carolina	108, 065	45. 8	49, 494	46. 8	50, 574	7. 4	7, 997
South Dakota	105, 838	57. 3	60, 645	37. 2	39, 372	5. 5	5, 821
Tennessee	229, 554	41. 2	94, 576	53. 3	122, 353	5. 5	12, 625
Texas	555, 763	47. 9	266, 210	44. 1	245, 092	8. 0	44, 461
Utah	48, 939	30. 6	14, 975	58. 9	28, 825	10. 5	5, 139
Virginia	169, 254	45. 1	76, 334	45. 3	76, 672	9. 6	16, 248
Washington	144, 962	50. 1	72, 626	43. 1	62, 479	6. 8	9, 857
West Virginia	143, 503	49. 1	70, 460	41. 7	59, 841	9. 2	13, 202
Wisconsin	304, 493	52. 9	161, 077	42. 0	127, 887	5. 1	15, 529
Wyoming	44, 885	69. 4	31, 150	28. 1	12, 613	2. 5	1, 122
Subtotal	7, 874, 544	45. 0	3, 545, 898	45. 6	3, 590, 604	9. 4	738, 042
Other States [1]	1, 417, 451	38. 1	540, 676	48. 8	691, 741	13. 1	185, 034
Totals and averages	9, 291, 995	44. 0	4, 086, 574	46. 1	4, 282, 345	9. 9	923, 076

[1] Includes Arkansas, Connecticut, Delaware, Georgia, Maine, New Jersey, New York, and Vermont.

Table 8.—*Vehicle-mileage of trucks and combinations with both origin and destination rural, with one rural and the other urban, and with both urban on ALL RURAL ROADS in each State in the year 1940*

State	Total truck travel	Origin and destination					
		Both rural		One rural—one urban		Both urban	
	Thousands of vehicle-miles	*Per cent*	*Thousands of vehicle-miles*	*Per cent*	*Thousands of vehicle-miles*	*Per cent*	*Thousands of vehicle-miles*
Alabama	575, 746	32. 2	185, 262	47. 9	275, 882	19. 9	114, 602
Arizona	199, 955	29. 1	58, 128	44. 6	89, 217	26. 3	52, 610
California	1, 743, 916	18. 2	317, 363	43. 8	763, 607	38. 0	662, 946
Colorado	453, 015	43. 9	108, 068	50. 4	228, 455	25. 7	116, 492
Florida	631, 059	22. 6	142, 248	43. 7	275, 937	33. 7	212, 874
Idaho	260, 865	32. 5	84, 741	49. 9	130, 154	17. 6	45, 970
Illinois	1, 200, 118	19. 7	236, 421	36. 2	434, 761	44. 1	528, 936
Indiana	1, 187, 582	23. 0	272, 640	38. 4	456, 699	38. 6	458, 243
Iowa	708, 096	23. 3	164, 769	49. 7	351, 833	27. 0	191, 494
Kansas	681, 676	27. 0	183, 638	42. 1	287, 245	30. 9	210, 793
Kentucky	547, 929	22. 2	121, 674	42. 9	235, 297	34. 9	190, 958
Louisiana	447, 498	24. 9	111, 409	42. 3	189, 387	32. 8	146, 702
Maryland	502, 062	13. 6	68, 278	40. 7	204, 272	45. 7	229, 512
Massachusetts	664, 673	15. 1	100, 515	36. 1	240, 153	48. 8	324, 005
Michigan	1, 088, 791	13. 0	141, 606	34. 3	373, 484	52. 7	573, 701
Minnesota	666, 071	30. 6	203, 606	44. 4	295, 590	25. 0	166, 875
Mississippi	772, 682	24. 6	190, 194	47. 6	367, 368	27. 8	215, 120
Missouri	873, 923	28. 7	250, 909	46. 8	408, 750	24. 5	214, 264
Montana	299, 898	33. 9	101, 582	50. 3	150, 836	15. 8	47, 480
Nebraska	410, 807	31. 6	129, 836	41. 6	170, 884	26. 8	110, 087
Nevada	75, 497	32. 2	24, 309	49. 4	37, 267	18. 4	13, 921
New Hampshire	146, 501	22. 1	32, 340	41. 2	60, 410	36. 7	53, 751
New Mexico	276, 166	37. 4	103, 360	42. 8	118, 201	19. 8	54, 605
North Carolina	838, 939	15. 6	130, 769	36. 3	304, 633	48. 1	403, 537
North Dakota	142, 025	38. 6	54, 786	46. 6	66, 145	14. 8	21, 094
Ohio	1, 365, 712	8. 0	109, 379	10. 7	463, 800	58. 0	792, 533
Oklahoma	694, 743	24. 0	166, 581	42. 3	293, 913	33. 7	234, 249
Oregon	329, 529	24. 4	80, 276	54. 4	179, 186	21. 2	70, 067
Pennsylvania	1, 421, 912	14. 9	212, 185	39. 4	559, 626	45. 7	650, 101
Rhode Island	64, 243	7. 6	4, 886	36. 2	23, 225	56. 2	36, 132

TABLE 8.—*Vehicle-mileage of trucks and combinations with both origin and destination rural, with one rural and the other urban, and with both urban on ALL RURAL ROADS in each State in the year 1940*—Continued

State	Total truck travel	Origin and destination					
		Both rural		One rural—one urban		Both urban	
	Thousands of vehicle-miles	*Percent*	*Thousands of vehicle-miles*	*Percent*	*Thousands of vehicle-miles*	*Percent*	*Thousands of vehicle-miles*
South Carolina	481, 960	17. 9	86, 510	42. 9	206, 488	39. 2	188, 962
South Dakota	247, 320	33. 1	81, 867	48. 6	120, 158	18. 3	45, 295
Tennessee	612, 204	23. 4	143, 173	48. 5	296, 841	28. 1	172, 190
Texas	1, 980, 100	21. 4	424, 311	45. 6	903, 136	33. 0	652, 653
Utah	159, 653	19. 5	31, 139	56. 8	90, 714	23. 7	37, 800
Virginia	752, 422	22. 2	167, 308	44. 3	333, 266	33. 5	251, 848
Washington	493, 280	21. 9	107, 806	48. 8	240, 818	29. 3	144, 656
West Virginia	390, 831	30. 3	118, 441	39. 2	153, 331	30. 5	119, 059
Wisconsin	797, 386	30. 2	240, 433	40. 8	325, 537	29. 0	231, 416
Wyoming	150, 627	28. 5	42, 993	41. 4	62, 312	30. 1	45, 322
Subtotal	25, 337, 412	21. 8	5, 535, 739	42. 5	10, 768, 818	35. 7	9, 032, 855
Other States [1]	4, 512, 889	18. 3	825, 821	40. 2	1, 814, 681	41. 5	1, 872, 387
Totals and averages	29, 850, 301	21. 3	6, 361, 560	42. 2	12, 583, 499	36. 5	10, 905, 242

[1] Includes Arkansas, Connecticut, Delaware, Georgia, Maine, New Jersey, New York, and Vermont.

movement. For example, fruit and vegetables are shipped directly by truck from rural areas in the south to the large cities of the north.

Figure 6, which was prepared from data in tables 6, 7, and 8, shows the percentage relations of the mileage of trucks between urban areas, between urban and rural areas, and between rural areas, on main and local roads, separately and combined. On both main and local roads the movement between rural and urban areas, or what might be called the farm-to-city and city-to-farm movement, was between 40 percent and 50 percent of the total. On main roads, the balance was mostly urban-urban with only about 11 percent rural-rural, while on local roads the movement between rural areas made up most of the balance, the city-to-city movement amounting to only about 10 percent.

Stated another way, most of the travel between cities took place on main roads, considerable travel between urban and rural areas took place on both main and local roads, and most of the travel between rural areas took place on local roads. Figure 7 shows the

TABLE 9.—*Percentage loaded, average carried load, and ton-mileage of carried load, of trucks and combinations, with both origin and destination rural, with one rural and the other urban, and with both urban, on MAIN RURAL ROADS in each State in the year 1940*

State	Origin and destination												Total ton-miles
	Both rural				One rural—one urban				Both urban				
	Percentage loaded	Carried load			Percentage loaded	Carried load			Percentage loaded	Carried load			
		Average weight	Ton-miles	Percentage of State total		Average weight	Ton-miles	Percentage of State total		Average weight	Ton-miles	Percentage of State total	
		Tons	*Thousands*			*Tons*	*Thousands*			*Tons*	*Thousands*		*Thousands*
Alabama	44. 4	2. 81	57, 727	11. 3	56. 4	2. 73	241, 189	47. 0	70. 5	3. 17	213, 968	41. 7	512, 884
Arizona	47. 1	2. 78	20, 450	6. 4	55. 2	2. 76	80, 552	25. 1	75. 5	6. 18	219, 618	68. 5	320, 620
California	66. 9	3. 94	229, 701	5. 4	73. 4	4. 30	1, 387, 196	32. 7	72. 1	6. 46	2, 624, 360	61. 9	4, 241, 257
Colorado	67. 4	2. 51	97, 123	14. 0	64. 1	2. 50	310, 117	44. 8	76. 9	3. 24	284, 953	41. 2	692, 223
Florida	36. 3	3. 08	74, 104	3. 5	46. 2	3. 38	306, 059	35. 2	57. 5	4. 36	489, 147	56. 3	869, 310
Idaho	54. 3	2. 29	46, 733	20. 8	59. 4	1. 88	98, 277	43. 8	70. 5	2. 63	79, 436	35. 4	224, 446
Illinois	55. 0	3. 33	105, 612	4. 9	58. 9	3. 10	519, 371	24. 0	71. 1	4. 27	1, 538, 364	71. 1	2, 163, 347
Indiana	60. 4	2. 74	135, 855	5. 8	64. 4	2. 65	481, 006	20. 4	77. 7	5. 16	1, 739, 302	73. 8	2, 356, 163
Iowa	65. 9	3. 57	125, 899	12. 2	66. 1	2. 55	401, 575	39. 1	78. 7	3. 58	500, 022	48. 7	1, 027, 496
Kansas	53. 0	2. 20	44, 069	4. 8	58. 0	2. 90	288, 531	31. 5	67. 1	4. 43	583, 746	63. 7	916, 346
Kentucky	55. 1	2. 85	74, 909	9. 3	63. 6	2. 96	332, 872	41. 6	76. 5	2. 80	393, 114	49. 1	800, 895
Louisiana	44. 9	2. 12	30, 861	6. 5	55. 2	2. 01	131, 894	27. 8	65. 6	3. 52	311, 435	65. 7	474, 190
Maryland	56. 8	2. 52	51, 730	4. 7	65. 1	3. 66	347, 113	31. 6	73. 4	4. 34	698, 929	63. 7	1, 097, 772
Massachusetts	37. 3	2. 03	14, 388	1. 7	79. 5	2. 42	107, 987	12. 5	74. 9	3. 73	740, 710	85. 8	863, 085
Michigan	50. 8	2. 44	73, 886	4. 1	64. 3	3. 11	425, 724	23. 6	73. 3	3. 62	1, 301, 139	72. 3	1, 800, 749
Minnesota	55. 6	2. 81	119, 305	13. 5	65. 6	2. 59	342, 836	38. 7	82. 5	3. 28	423, 132	47. 8	885, 273
Mississippi	51. 3	2. 35	90, 585	11. 9	58. 6	2. 46	331, 655	43. 7	68. 4	2. 51	335, 491	44. 2	758, 731
Missouri	51. 8	2. 92	226, 691	15. 7	62. 3	2. 55	532, 601	37. 0	70. 7	4. 64	682, 156	47. 3	1, 441, 448
Montana	48. 7	2. 55	43, 215	12. 8	47. 5	3. 71	182, 585	54. 2	63. 7	3. 96	110, 998	33. 0	336, 798
Nebraska	57. 9	2. 68	60, 791	12. 4	68. 0	2. 32	179, 768	36. 7	73. 6	3. 34	248, 948	50. 9	489, 507
Nevada	62. 8	3. 70	22, 796	19. 1	60. 6	2. 91	46, 866	39. 2	72. 2	5. 33	49, 771	41. 7	119, 433
New Hampshire	61. 3	3. 17	25, 208	12. 7	70. 1	2. 45	66, 209	33. 4	80. 6	2. 88	107, 026	53. 9	198, 443
New Mexico	53. 2	2. 45	44, 580	15. 2	55. 8	2. 55	110, 270	37. 7	74. 4	3. 60	137, 769	47. 1	292, 619
North Carolina	48. 9	1. 32	36, 609	2. 1	59. 3	1. 84	252, 498	14. 5	72. 3	5. 18	1, 453, 664	83. 4	1, 742, 774
North Dakota	55. 2	3. 67	55, 106	26. 4	69. 8	2. 99	100, 974	48. 4	77. 9	3. 28	52, 478	25. 2	208, 558
Ohio	53. 5	3. 23	25, 613	0. 7	54. 7	3. 39	464, 831	14. 5	65. 6	5. 74	2, 724, 673	84. 8	3, 215, 117
Oklahoma	53. 2	2. 94	87, 948	9. 1	55. 1	2. 86	328, 561	34. 0	65. 6	3. 73	549, 099	56. 9	965, 698
Pennsylvania	53. 4	3. 00	129, 251	5. 8	59. 8	2. 84	651, 987	29. 2	73. 4	3. 30	1, 450, 562	65. 0	2, 231, 800
Rhode Island	55. 2	2. 18	2, 492	2. 7	64. 1	2. 73	29, 488	31. 5	67. 2	2. 69	61, 552	65. 8	93, 532
South Carolina	56. 6	3. 31	69, 367	7. 2	63. 2	3. 58	352, 879	36. 4	70. 3	4. 29	545, 832	56. 4	968, 078
South Dakota	68. 4	2. 41	34, 933	16. 7	57. 8	1. 87	87, 179	41. 8	76. 6	2. 86	86, 577	41. 5	208, 689
Tennessee	46. 4	2. 40	54, 266	8. 0	60. 8	2. 43	257, 273	37. 8	74. 7	3. 09	369, 061	54. 2	680, 600
Texas	55. 4	2. 32	202, 953	10. 2	57. 3	2. 04	770, 840	38. 6	66. 1	2. 54	1, 020, 516	51. 2	1, 994, 309
Utah	57. 5	3. 09	28, 749	13. 8	60. 0	2. 95	109, 470	52. 3	75. 5	2. 88	70, 947	33. 9	209, 166
Virginia	52. 5	3. 31	157, 936	11. 5	66. 8	2. 86	490, 425	35. 9	72. 5	4. 21	718, 199	52. 6	1, 366, 560
Washington	47. 7	3. 54	58, 851	7. 6	56. 3	2. 81	282, 332	36. 6	70. 6	4. 52	430, 727	55. 8	771, 910
West Virginia	52. 3	2. 83	71, 004	17. 8	57. 7	2. 09	112, 612	28. 2	68. 0	3. 00	215, 748	54. 0	399, 364
Wisconsin	62. 3	2. 15	106, 401	9. 3	70. 8	2. 37	331, 153	29. 1	83. 0	3. 92	702, 507	61. 6	1, 140, 061
Wyoming	54. 7	3. 54	22, 908	9. 1	52. 4	3. 88	101, 158	40. 1	65. 1	4. 46	128, 276	50. 8	252, 342
Subtotal	54. 3	2. 79	2, 960, 605	7. 5	61. 2	2. 78	11, 975, 943	30. 5	71. 5	4. 15	24, 394, 932	62. 0	39, 331, 480
Other States [1]	48. 9	3. 08	483, 625	6. 6	60. 3	2. 94	2, 200, 331	29. 9	72. 3	3. 68	4, 670, 285	63. 5	7, 354, 241
Totals and averages	53. 5	2. 83	3, 444, 230	7. 4	61. 1	2. 80	14, 176, 274	30. 3	71. 6	4. 06	29, 065, 217	62. 3	46, 685, 721

[1] Includes Arkansas, Connecticut, Delaware, Georgia, Maine, New Jersey, New York, Oregon, and Vermont.

vehicle-miles in each of the three origin and destination categories, divided between main and local roads, in each case. For all rural roads, main and local combined, the truck-mileage between rural and urban areas amounted to about 12.6 billion, compared to about 10.9 billion for urban-urban travel and 6.4 billion for rural-rural travel.

Though the truck mileage from city to city was less than that between rural and urban areas, and little more than a third of the total for all three classifications, the ton-mileage hauled between cities was greater than that of the other two origin and destination classifications combined. This is because a greater proportion of the trucks operating between cities were loaded, and the average load was greater than for other trucks. Figure 8 shows that 72.2 percent of the trucks operating between cities were loaded, compared to 63.4 percent of those operating between urban and rural areas and 56.2 percent of those operating between rural areas. Figure 9 and table 11 show that the average carried load was about 4.0 tons for the urban-urban trucks, 2.5 for the rural-urban and urban-rural trucks, and 2.3 tons for the rural-rural trucks. The resulting ton-miles of carried load were 31.1 billion for the urban-urban, 19.9 billion for the urban-rural and rural-urban, and 8.1 billion for the rural-rural classifications, as indicated by the areas of the rectangles in figure 9.

TRUCKING CLASSIFIED AS INTRASTATE, INTERSTATE, AND TRANSSTATE

The classification of origins and destinations on the basis of location within or without the State in which the vehicle was observed is helpful from the point of view of jurisdiction or extent of interest. A trip occurring wholly within a State would presumably be of interest mainly to the State in which it occurred, while one across a state line would be of interest to two States and to the Federal Government. This classification also gives some indication of trip extent and aids in evaluating the need for through routes.

TABLE 10.—*Percentage loaded, average carried load, and ton-mileage of carried load, of trucks and combinations, with both origin and destination rural, with one rural and the other urban, and with both urban, on LOCAL RURAL ROADS in each State in the year 1940*

State	Origin and destination												Total ton-miles
	Both rural				One rural—one urban				Both urban				
		Carried load				Carried load				Carried load			
	Percentage loaded	Average weight	Ton-miles	Percentage of State total	Percentage loaded	Average weight	Ton-miles	Percentage of State total	Percentage loaded	Average weight	Ton-miles	Percentage of State total	
		Tons	*Thousands*			*Tons*	*Thousands*			*Tons*	*Thousands*		*Thousands*
Alabama	50.9	2.21	156,133	43.3	64.7	2.15	166,718	46.2	80.9	2.48	37,937	10.5	360,788
Arizona	54.3	2.32	53,437	40.8	63.8	2.28	52,764	40.2	87.1	5.14	24,930	19.0	131,131
California	64.3	2.21	328,096	27.9	70.6	2.42	553,401	47.1	81.3	3.63	293,083	25.0	1,174,580
Colorado	69.6	2.02	71,376	58.5	66.1	2.01	46,298	37.9	79.4	2.61	4,355	3.6	122,029
Florida	40.3	2.24	68,524	33.3	51.4	2.46	100,693	48.9	63.8	3.18	36,637	17.8	205,854
Idaho	57.6	1.83	49,634	52.4	63.0	1.50	39,956	42.2	79.9	2.09	5,104	5.4	94,694
Illinois	61.4	2.31	253,987	48.8	65.8	2.16	213,609	41.0	79.4	2.97	53,089	10.2	520,685
Indiana	63.7	1.65	200,172	44.2	67.7	1.60	190,272	42.0	82.0	3.11	62,639	13.8	453,083
Iowa	69.2	1.94	149,539	53.2	69.5	1.38	109,104	38.8	82.7	1.95	22,677	8.0	281,320
Kansas	58.1	1.43	121,207	41.6	63.7	1.90	139,588	48.0	73.6	2.87	30,323	10.4	291,118
Kentucky	62.7	2.19	101,506	48.0	72.4	2.28	96,535	45.6	87.2	2.15	13,616	6.4	211,657
Louisiana	53.2	1.90	79,869	42.6	61.3	1.80	77,609	41.4	80.1	3.15	30,160	16.0	187,638
Maryland	62.1	2.34	46,874	33.3	71.2	2.83	60,132	42.7	80.3	4.04	33,833	24.0	140,839
Massachusetts	40.0	1.78	57,864	10.6	85.3	2.13	334,799	61.1	80.4	3.27	155,268	28.3	547,931
Michigan	56.2	1.20	55,468	16.3	71.1	1.53	174,800	51.4	74.1	1.78	109,512	32.3	339,780
Minnesota	62.8	2.00	159,361	51.2	74.2	1.84	128,688	41.3	93.2	2.33	23,195	7.5	311,244
Mississippi	55.8	1.61	103,315	45.3	63.7	1.48	100,006	43.9	74.4	1.72	24,560	10.8	227,881
Missouri	55.4	2.12	118,310	50.6	66.6	2.06	99,824	42.7	75.5	3.36	15,534	6.7	233,668
Montana	50.6	2.03	68,480	47.4	49.2	2.96	68,710	47.5	66.2	3.16	7,362	5.1	144,552
Nebraska	63.4	1.88	108,025	55.7	74.4	1.63	69,036	35.6	80.6	2.34	16,850	8.7	193,911
Nevada	62.8	2.32	21,068	58.8	60.6	1.91	12,340	34.5	72.1	3.34	2,395	6.7	35,803
New Hampshire	66.5	2.56	32,991	40.4	76.0	1.99	33,031	40.5	87.2	2.33	15,592	19.1	81,614
New Mexico	58.8	2.03	82,184	57.6	61.7	2.11	52,864	37.0	82.3	2.98	7,753	5.4	142,801
North Carolina	56.4	1.40	58,241	25.8	68.4	1.95	97,795	43.3	83.4	5.49	69,912	30.9	225,948
North Dakota	59.7	2.55	41,982	59.1	75.4	2.08	27,988	39.4	84.1	2.28	1,056	1.5	71,026
Ohio	56.0	2.49	131,607	20.0	57.3	2.61	319,880	48.5	68.7	4.42	208,013	31.5	659,500
Oklahoma	55.2	1.79	108,254	51.8	57.1	1.76	85,469	40.9	68.0	2.29	15,330	7.3	209,053
Pennsylvania	58.2	2.13	163,240	32.3	69.1	2.01	244,798	48.4	80.2	2.35	97,301	19.3	505,339
Rhode Island	57.7	1.62	2,639	18.6	67.1	2.02	8,648	61.1	70.1	2.00	2,867	20.3	14,154
South Carolina	61.7	2.03	62,123	40.1	69.0	2.20	76,666	49.5	76.6	2.64	16,156	10.4	154,945
South Dakota	65.3	1.95	77,071	62.0	62.2	1.53	37,476	30.1	73.1	2.32	9,854	7.9	124,401
Tennessee	53.3	1.91	96,273	33.5	69.7	1.93	164,733	57.3	85.8	2.46	26,661	9.2	287,667
Texas	58.2	1.61	249,626	48.6	60.2	1.42	209,725	40.8	69.5	1.76	54,495	10.6	513,846
Utah	60.2	2.10	18,926	30.0	62.8	2.00	36,230	57.4	79.1	1.95	7,934	12.6	63,090
Virginia	56.9	2.28	99,279	40.3	72.4	1.98	109,783	44.6	78.6	2.91	37,105	15.1	246,167
Washington	54.2	2.41	94,973	48.2	64.0	1.93	77,324	39.3	80.3	3.11	24,623	12.5	196,920
West Virginia	56.5	2.12	84,359	50.8	62.3	1.61	59,950	36.1	73.5	2.24	21,770	13.1	166,079
Wisconsin	68.3	1.54	169,363	44.9	77.5	1.69	168,023	44.6	90.9	2.81	39,624	10.5	377,010
Wyoming	56.6	2.60	45,989	67.6	54.4	2.86	19,620	28.8	67.4	3.27	2,476	3.6	68,085
Subtotal	58.6	1.94	3,991,365	38.7	67.1	1.96	4,664,885	45.2	77.6	2.92	1,661,581	16.1	10,317,831
Other States [1]	52.0	2.04	620,119	30.2	71.6	1.97	1,042,306	50.7	79.9	2.61	392,367	19.1	2,054,792
Totals and averages	57.7	1.96	4,611,484	37.3	67.9	1.96	5,707,191	46.1	78.0	2.85	2,053,948	16.6	12,372,623

[1] Includes Askansas, Connecticut, Delaware, Georgia, Maine, New Jersey, New York, Oregon, and Vermont.

In tables 12 to 17, inclusive, the term "intrastate" describes movements with both origin and destination within a single State, "interstate" describes movements with either origin or destination, but not both, in the State for which data are given, and "transstate" describes movements with neither origin nor destination in the State for which data are given. It is to be noted that "interstate" traffic, according to these definitions, does not include "transstate" traffic as it does in the common usage of the term.

For correct understanding of the origin and destination tables it must be borne in mind that vehicle-mileages in different categories are computed by multiplying the average number of vehicles in each category passing over a section of road, within a given period of time, by the length of the road section, and not by considering individual trips. The same vehicle on a given trip may have been differently classified in different States. Thus, if a vehicle makes a trip originating in one State, crossing another State and ending in a third State, the travel in the first and third States would be classed as interstate and that in the intervening State would be classed as transstate.

The distinction between interstate traffic and transstate traffic is important to individual States because interstate traffic results in imports and exports, whereas transstate traffic, except in cases of the taking on or putting off of partial loads en route, contributes nothing to the commerce of the State and yet must be accommodated on its highways. From a national point of view the distinction is of less importance and, for many purposes, the two classes may well be combined in considering the national totals. They do, however, tend to provide a rough division on the basis of trip extent since all of the transstate travel is by trucks on trips extending into at least three States, whereas the interstate travel is largely by trucks extending into only two States, though it includes some travel of greater extent.

The travel by trucks across States in which the trip neither begins nor ends is not as great as might be supposed. In only 6 of the 41 States shown separately in table 12 did it amount to as much as 10 percent of the total truck travel on main roads. These were New

TABLE 11.—*Percentage loaded, average carried load, and ton-mileage of carried load, of trucks and combinations with both origin and destination rural, with one rural and the other urban, and with both urban, on* ALL RURAL ROADS *in each State in the year 1940*

State	Origin and destination												Total ton-miles
	Both rural				One rural—one urban				Both urban				
	Percentage loaded	Carried load			Percentage loaded	Carried load			Percentage loaded	Carried load			
		Average weight	Ton-miles	Percentage of State total		Average weight	Ton-miles	Percentage of State total		Average weight	Ton-miles	Percentage of State total	
		Tons	*Thousands*			*Tons*	*Thousands*			*Tons*	*Thousands*		*Thousands*
Alabama	49. 3	2. 34	213, 860	24. 5	60. 0	2. 46	407, 907	46. 7	72. 3	3. 04	251, 905	28. 8	873, 672
Arizona	52. 3	2. 43	73, 887	16. 4	58. 7	2. 55	133, 316	29. 5	76. 7	6. 06	244, 548	54. 1	451, 751
California	65. 0	2. 70	557, 797	10. 3	72. 2	3. 52	1, 940, 597	35. 8	73. 5	5. 99	2, 917, 443	53. 9	5, 415, 837
Colorado	68. 5	2. 28	168, 499	20. 7	64. 4	2. 42	356, 445	43. 8	76. 9	3. 23	289, 308	35. 5	814, 252
Florida	38. 4	2. 61	142, 628	13. 3	47. 7	3. 09	406, 752	37. 8	58. 1	4. 25	525, 784	48. 9	1, 075, 164
Idaho	56. 1	2. 03	96, 367	30. 2	60. 6	1. 75	138, 233	43. 3	71. 1	2. 59	84, 540	26. 5	319, 140
Illinois	59. 8	2. 54	359, 599	13. 4	61. 3	2. 75	732, 980	27. 3	71. 4	4. 21	1, 591, 453	59. 3	2, 684, 032
Indiana	62. 7	1. 97	336, 027	12. 0	65. 8	2. 24	671, 278	23. 9	77. 9	5. 05	1, 801, 941	64. 1	2, 809, 246
Iowa	68. 2	2. 45	275, 438	21. 1	67. 2	2. 16	510, 679	39. 0	79. 0	3. 46	522, 699	39. 9	1, 308, 816
Kansas	57. 1	1. 58	165, 276	13. 7	60. 3	2. 47	428, 119	35. 4	67. 6	4. 31	614, 069	50. 9	1, 207, 464
Kentucky	59. 7	2. 43	176, 415	17. 4	65. 8	2. 77	429, 407	42. 4	76. 9	2. 77	406, 730	40. 2	1, 012, 552
Louisiana	50. 8	1. 96	110, 730	16. 7	57. 5	1. 92	209, 503	31. 7	66. 8	3. 49	341, 595	51. 6	661, 828
Maryland	59. 3	2. 43	98, 604	8. 0	66. 0	3. 02	407, 245	32. 9	73. 7	4. 33	732, 762	59. 1	1, 238, 611
Massachusetts	39. 5	1. 82	72, 252	5. 1	84. 0	2. 20	442, 786	31. 4	75. 9	3. 64	895, 978	63. 5	1, 411, 016
Michigan	54. 0	1. 69	129, 354	6. 0	67. 2	2. 39	600, 524	28. 1	73. 4	3. 35	1, 410, 651	65. 9	2, 140, 529
Minnesota	60. 1	2. 28	278, 666	23. 3	68. 4	2. 33	471, 524	39. 4	83. 2	3. 22	446, 327	37. 3	1, 196, 517
Mississippi	54. 0	1. 89	193, 900	19. 6	60. 1	1. 96	431, 661	43. 8	69. 0	2. 43	361, 051	36. 6	986, 612
Missouri	53. 2	2. 58	345, 001	20. 6	63. 1	2. 45	632, 425	37. 7	70. 8	4. 60	697, 690	41. 7	1, 675, 116
Montana	50. 0	2. 20	111, 695	23. 2	48. 0	3. 47	251, 295	52. 2	63. 9	3. 90	118, 360	24. 6	481, 350
Nebraska	61. 8	2. 10	168, 816	24. 7	70. 1	2. 08	248, 804	36. 4	74. 2	3. 26	265, 798	38. 9	683, 418
Nevada	62. 8	2. 87	43, 864	28. 3	60. 6	2. 62	59, 206	38. 1	72. 2	5. 19	52, 166	33. 6	155, 236
New Hampshire	64. 4	2. 79	58, 199	20. 8	72. 2	2. 27	99, 240	35. 4	81. 6	2. 80	122, 618	43. 8	280, 057
New Mexico	56. 9	2. 16	126, 764	29. 1	57. 8	2. 39	163, 134	37. 5	74. 9	3. 56	145, 522	33. 4	435, 420
North Carolina	53. 2	1. 36	94, 850	4. 8	61. 5	1. 87	350, 293	17. 8	72. 7	5. 19	1, 523, 576	77. 4	1, 968, 719
North Dakota	57. 5	3. 08	97, 088	34. 7	71. 3	2. 73	128, 962	46. 1	78. 0	3. 25	53, 534	19. 2	279, 584
Ohio	55. 6	2. 58	157, 220	4. 0	55. 9	3. 03	784, 711	20. 3	65. 9	5. 62	2, 932, 686	75. 7	3, 874, 617
Oklahoma	54. 5	2. 16	196, 202	16. 7	55. 7	2. 53	414, 030	35. 2	65. 7	3. 67	564, 429	48. 1	1, 174, 661
Pennsylvania	56. 4	2. 44	292, 491	10. 7	62. 7	2. 55	896, 785	32. 8	74. 0	3. 22	1, 547, 863	56. 5	2, 737, 139
Rhode Island	56. 6	1. 85	5, 131	4. 8	64. 9	2. 53	38, 136	35. 4	67. 4	2. 65	64, 419	59. 8	107, 686
South Carolina	59. 5	2. 55	131, 490	11. 7	64. 7	3. 22	429, 545	38. 2	70. 5	4. 22	561, 988	50. 1	1, 123, 023
South Dakota	66. 1	2. 08	112, 004	33. 6	59. 2	1. 75	124, 655	37. 4	76. 1	2. 80	96, 411	29. 0	333, 070
Tennessee	51. 0	2. 06	150, 539	15. 5	64. 5	2. 20	422, 006	43. 6	75. 6	3. 04	395, 722	40. 9	968, 267
Texas	57. 1	1. 87	452, 579	18. 0	58. 1	1. 87	980, 565	39. 1	66. 4	2. 48	1, 075, 011	42. 9	2, 508, 155
Utah	58. 8	2. 61	47, 675	17. 5	60. 9	2. 64	145, 700	53. 5	76. 0	2. 75	78, 881	29. 0	272, 256
Virginia	54. 5	2. 82	257, 215	15. 9	68. 1	2. 64	600, 208	37. 3	72. 9	4. 12	755, 304	46. 8	1, 612, 727
Washington	52. 1	2. 74	153, 824	15. 9	58. 3	2. 56	359, 656	37. 1	71. 3	4. 41	455, 350	47. 0	968, 830
West Virginia	54. 8	2. 39	155, 363	27. 5	59. 5	1. 89	172, 562	30. 5	68. 6	2. 91	237, 518	42. 0	565, 443
Wisconsin	66. 3	1. 73	275, 764	18. 2	73. 4	2. 09	499, 176	32. 9	83. 5	3. 84	742, 131	48. 9	1, 517, 071
Wyoming	56. 1	2. 86	68, 897	21. 5	52. 8	3. 67	120, 778	37. 7	65. 1	4. 43	130, 752	40. 8	320, 427
Subtotal	57. 1	2. 23	6, 951, 970	14. 0	63. 2	2. 49	16, 640, 828	33. 5	72. 0	4. 04	26, 056, 513	52. 5	49, 649, 311
Other States [1]	50. 9	2. 39	1, 103, 744	11. 7	64. 5	2. 52	3, 212, 637	34. 5	73. 0	3. 57	5, 062, 652	53. 8	9, 409, 033
Totals and averages	56. 2	2. 25	8, 055, 714	13. 6	63. 4	2. 49	19, 883, 465	33. 7	72. 2	3. 95	31, 119, 165	52. 7	59, 058, 344

[1] Includes Arkansas, Connecticut, Delaware, Georgia, Maine, New Jersey, New York, Oregon, and Vermont.

Table 12.—*Vehicle-mileage of trucks and combinations which traveled intrastate, interstate, and transstate on MAIN RURAL ROADS in each State in the year 1940*

State	Total truck travel	Intrastate		Interstate		Transstate	
	Thousands of vehicle-miles	*Percent*	*Thousands of vehicle-miles*	*Percent*	*Thousands of vehicle-miles*	*Percent*	*Thousands of vehicle-miles*
Alabama	298, 228	76. 9	229, 337	20. 5	61, 137	2. 6	7, 754
Arizona	115, 580	72. 3	83, 564	20. 9	24, 156	6. 8	7, 860
California	1, 090, 056	96. 3	1, 049, 724	3. 6	39, 242	. 1	1, 090
Colorado	365, 467	79. 0	288, 719	20. 4	74, 555	. 6	2, 193
Florida	457, 276	82. 7	378, 167	17. 1	78, 194	. 2	915
Idaho	168, 295	84. 3	141, 873	13. 9	23, 393	1. 8	3, 029
Illinois	848, 272	67. 4	571, 735	26. 0	220, 551	6. 6	55, 986
Indiana	797, 144	63. 0	502, 201	24. 0	191, 314	13. 0	103, 629
Iowa	469, 331	74. 8	351, 059	20. 2	94, 805	5. 0	23, 467
Kansas	405, 892	63. 2	256, 524	31. 3	127, 044	5. 5	22, 324
Kentucky	408, 204	71. 5	291, 866	19. 7	80, 416	8. 8	35, 922
Louisiana	286, 112	81. 0	231, 751	17. 6	50, 356	1. 4	4, 005
Maryland	429, 549	48. 0	206, 183	40. 4	173, 538	11. 6	49, 828
Massachusetts	340, 091	74. 4	253, 028	23. 7	80, 601	1. 9	6, 462
Michigan	763, 041	87. 0	663, 617	13. 0	99, 119	[1] . 0	305
Minnesota	433, 872	84. 0	364, 453	14. 7	63, 779	1. 3	5, 640
Mississippi	532, 310	76. 7	408, 282	18. 2	96, 880	5. 1	27, 148
Missouri	693, 801	71. 9	498, 843	24. 1	167, 206	4. 0	27, 752
Montana	182, 387	86. 4	157, 582	12. 1	22, 069	1. 5	2, 736
Nebraska	254, 168	76. 6	194, 692	20. 6	52, 359	2. 8	7, 117
Nevada	49, 341	75. 0	37, 006	16. 0	7, 895	9. 0	4, 440
New Hampshire	97, 620	58. 4	57, 010	29. 0	28, 310	12. 6	12, 300
New Mexico	163, 318	69. 2	113, 016	22. 2	36, 257	8. 6	14, 045
North Carolina	676, 412	72. 9	493, 104	20. 5	138, 665	6. 6	44, 643
North Dakota	95, 990	84. 6	81, 207	12. 7	12, 191	2. 7	2, 592
Ohio	989, 042	70. 5	697, 275	23. 6	233, 414	5. 9	58, 353
Oklahoma	490, 005	72. 4	354, 764	24. 9	122, 011	2. 7	13, 230
Oregon	235, 643	88. 0	207, 366	10. 6	24, 978	1. 4	3, 299
Pennsylvania	1, 062, 878	77. 1	819, 479	19. 2	204, 073	3. 7	39, 326
Rhode Island	53, 016	59. 8	31, 704	33. 8	17, 919	6. 4	3, 393
South Carolina	373, 895	63. 1	235, 928	24. 0	89, 735	12. 9	48, 232
South Dakota	141, 482	75. 1	106, 253	23. 9	33, 814	1. 0	1, 415
Tennessee	382, 650	76. 4	292, 345	19. 1	73, 086	4. 5	17, 219
Texas	1, 424, 337	92. 0	1, 310, 390	7. 5	106, 825	. 5	7, 122
Utah	110, 714	88. 5	97, 982	10. 2	11, 293	1. 3	1, 439
Vermont	65, 979	70. 3	46, 383	26. 4	17, 419	3. 3	2, 177
Virginia	583, 168	63. 2	368, 562	25. 8	150, 457	11. 0	64, 149
Washington	348, 318	87. 6	305, 127	11. 5	40, 056	. 9	3, 135
West Virginia	247, 328	76. 2	188, 464	19. 3	47, 734	4. 5	11, 130
Wisconsin	492, 893	81. 4	401, 215	15. 7	77, 384	2. 9	14, 294
Wyoming	105, 742	60. 6	64, 080	26. 4	27, 916	13. 0	13, 746
Subtotal	17, 528, 847	76. 6	13, 431, 860	19. 0	3, 322, 146	4. 4	774, 841
Other States [2]	3, 029, 459	67. 3	2, 039, 959	26. 4	798, 884	6. 3	190, 616
Totals and averages	20, 558, 306	75. 3	15, 471, 819	20. 0	4, 121, 030	4. 7	965, 457

[1] Less than 0.05 percent.

[2] Includes Arkansas, Connecticut, Delaware, Georgia, Maine, New Jersey, and New York.

Hampshire, Maryland, Virginia, and South Carolina along the eastern seaboard where there is much long-distance hauling north and south; Indiana which is crossed by most of the traffic between Chicago and the south and east, and Wyoming through which much of the hauling between the Pacific coast and the east or midwest funnels.

For the United States as a whole, this transstate travel amounted to only 4.7 percent of the total truck travel on main roads, 0.2 percent of that on local roads, and 3.3 percent of that on main and local roads combined (fig. 10). In fact, about 82 percent of the truck travel on all rural roads was confined to trips not extending beyond State borders. The annual vehicle-mileage of intrastate, interstate, and transstate truck

Table 13.—*Vehicle-mileage of trucks and combinations which traveled intrastate, interstate, and transstate on LOCAL RURAL ROADS in each State in the year 1940*

State	Total truck travel	Intrastate		Interstate		Transstate	
	Thousands of vehicle-miles	*Percent*	*Thousands of vehicle-miles*	*Percent*	*Thousands of vehicles-miles*	*Percent*	*Thousands of vehicle-miles*
Idaho	92, 570	93. 6	86, 646	6. 2	5, 739	0. 2	185
Indiana	390, 438	97. 0	378, 725	2. 9	11, 323	. 1	390
Iowa	238, 765	96. 6	230, 647	3. 3	7, 879	. 1	239
Michigan	325, 750	97. 4	317, 280	2. 5	8, 144	. 1	326
Minnesota	232, 199	97. 5	226, 394	2. 4	5, 573	. 1	232
Nebraska	156, 639	96. 5	151, 157	3. 3	5, 169	. 2	313
Nevada	26, 156	94. 8	24, 796	5. 1	1, 334	. 1	26
New Hampshire	48, 881	82. 7	40, 424	15. 3	7, 479	2. 0	978
New Mexico	112, 848	97. 8	110, 365	2. 1	2, 370	. 1	113
Ohio	376, 670	96. 7	364, 240	3. 1	11, 677	. 2	753
Oregon	93, 886	98. 9	92, 853	1. 1	1, 033	[1] . 0	....
South Carolina	108, 065	95. 9	103, 635	3. 8	4, 106	. 3	324
South Dakota	105, 838	92. 7	98, 112	7. 2	7, 620	. 1	106
Tennessee	229, 551	96. 4	221, 290	3. 4	7, 805	. 2	459
Utah	48, 939	97. 6	47, 764	2. 4	1, 175	[1] . 0	....
West Virginia	143, 503	93. 7	134, 462	5. 9	8, 467	. 4	574
Wisconsin	304, 493	97. 5	296, 881	2. 4	7, 308	. 1	304
Wyoming	44, 885	94. 6	42, 461	5. 0	2, 244	. 4	180
Subtotal	3, 080, 079	96. 3	2, 968, 132	3. 5	106, 445	. 2	5, 502
Other States [2]	6, 211, 916	96. 3	5, 979, 994	3. 5	220, 604	. 2	11, 318
Totals and averages	9, 291, 995	96. 3	8, 948, 126	3. 5	327, 049	. 2	16, 820

[1] Less than 0.05 percent.

[2] Includes Alabama, Arizona, Arkansas, California, Colorado, Connecticut, Delaware, Florida, Georgia, Illinois, Kansas, Kentucky, Louisiana, Maine, Maryland, Massachusetts, Mississippi, Missouri, Montana, New Jersey, New York, North Carolina, North Dakota, Oklahoma, Pennsylvania, Rhode Island, Texas, Vermont, Virginia, and Washington.

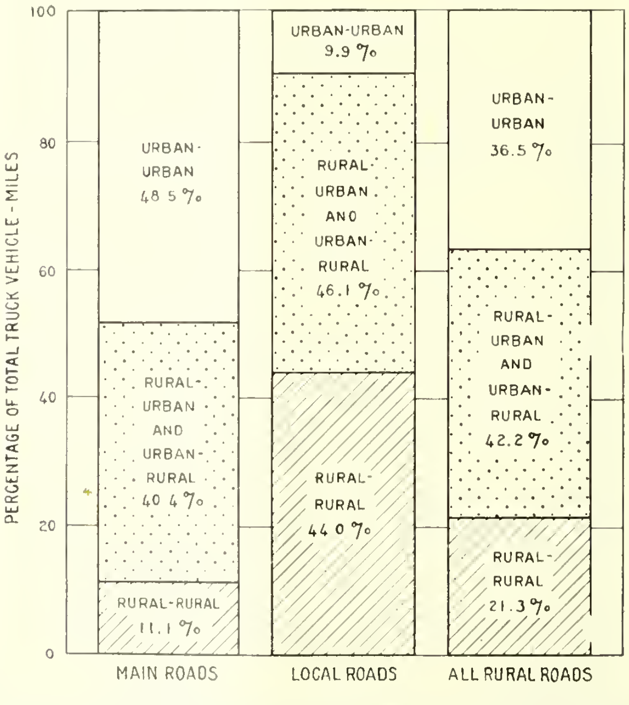

Figure 6.—Percentage Distribution of Vehicle-Miles of Trucks on the Basis of Rural and Urban Origins and Destinations on Main and Local Rural Roads in the Year 1940.

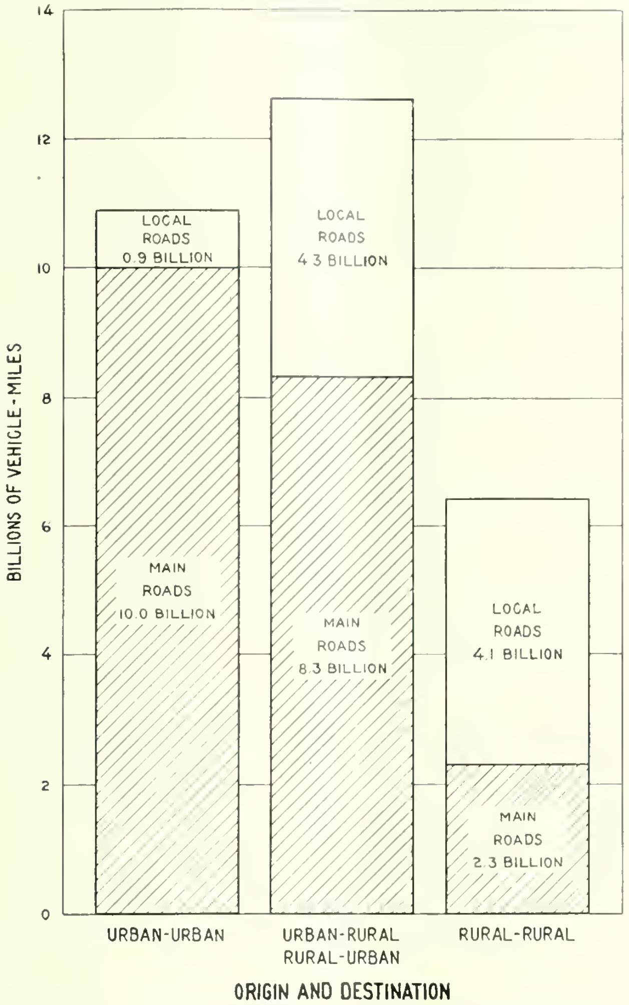

Figure 7.—Vehicle-Miles of Trucks With Origin and Destination Both Urban, With One Urban and the Other Rural, and With Both Rural, on Main and Local Rural Roads in the Year 1940.

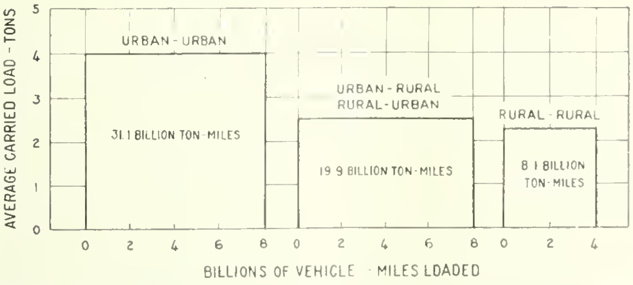

Figure 9.—Ton-Miles of Load Carried by Trucks With Origin and Destination Both Urban, With One Urban and the Other Rural, and With Both Rural on All Rural Roads in the Year 1940.

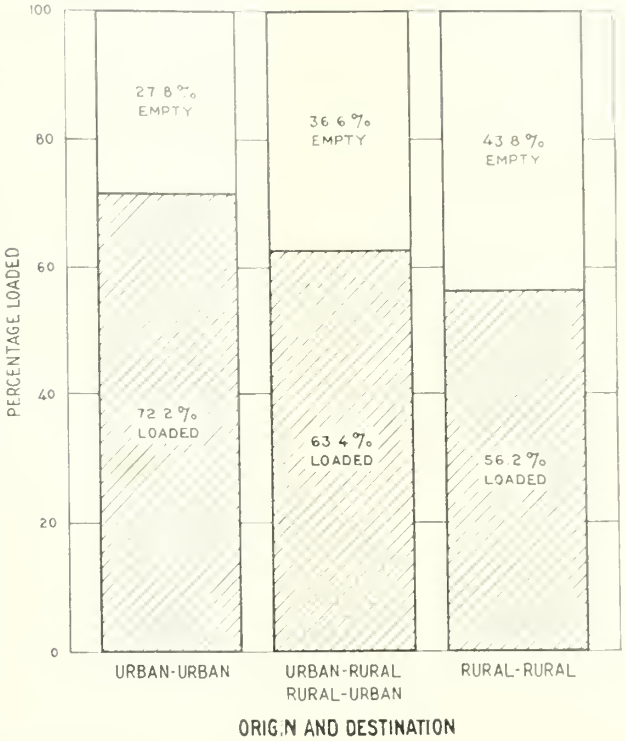

Figure 8. Loaded and Empty Trucks as Percentages of Trucks With Origin and Destination Both Urban, With One Urban and the Other Rural, and With Both Rural, on All Rural Roads in the Year 1940.

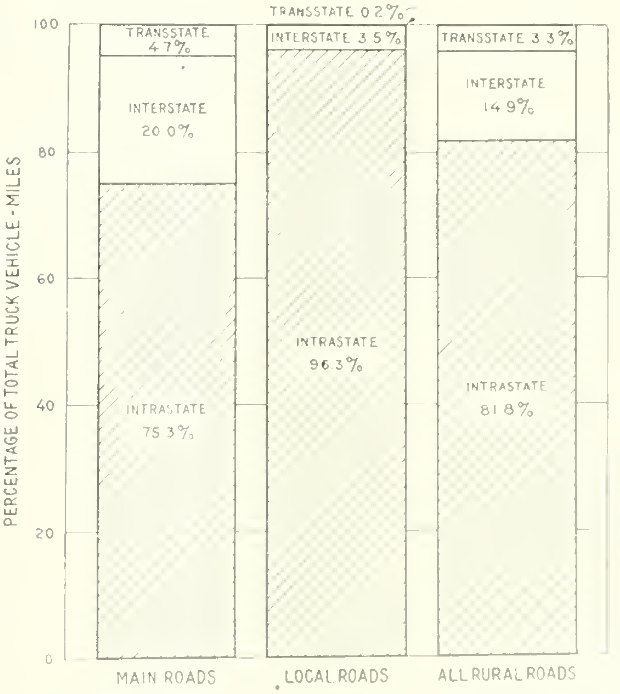

Figure 10. Percentage Distribution of Vehicle-Miles of Trucks on the Basis of Intrastate, Interstate, and Transstate Trip Classifications, on Main and Local Rural Roads in the Year 1940.

travel, shown separately for main and local roads, in each case, is given in figure 11.

The interstate and transstate movement of commodities in ton-miles was of much larger proportions than would appear from consideration of the vehicle-mileage

TABLE 14.—*Vehicle-mileage of trucks and combinations which traveled intrastate, interstate, and transstate on ALL RURAL ROADS in each State in the year 1940*

State	Total truck travel	Intrastate		Interstate		Transstate	
	Thousands of vehicle-miles	*Per-cent*	*Thousands of vehicle-miles*	*Per-cent*	*Thousands of vehicle-miles*	*Per-cent*	*Thousands of vehicle-miles*
Alabama	575,746	86.2	496,587	12.4	71,127	1.4	8,032
Arizona	199,955	82.4	164,648	13.6	27,278	4.0	8,029
California	1,743,916	97.4	1,699,661	2.5	43,165	.1	1,090
Colorado	453,015	82.3	373,115	17.2	77,707	.5	2,193
Florida	631,059	86.7	516,737	13.2	83,407	.1	915
Idaho	260,865	87.6	228,519	11.2	29,132	1.2	3,214
Illinois	1,200,118	75.6	906,692	19.7	236,736	4.7	56,690
Indiana	1,187,582	74.2	880,926	17.1	202,637	8.7	104,019
Iowa	708,096	82.2	581,706	14.5	102,684	3.3	23,706
Kansas	681,676	75.7	516,313	20.9	142,488	3.4	22,875
Kentucky	547,929	77.8	426,282	15.6	85,306	6.6	36,341
Louisiana	447,498	86.7	388,134	12.4	55,359	.9	4,005
Maryland	502,062	54.4	273,185	35.6	178,759	10.0	50,118
Massachusetts	664,673	84.8	563,653	14.2	94,233	1.0	6,787
Michigan	1,088,791	90.1	980,897	9.8	107,263	.1	631
Minnesota	666,071	88.7	590,847	10.4	69,352	.9	5,872
Mississippi	772,682	82.9	640,481	13.5	104,572	3.6	27,629
Missouri	873,923	76.8	671,040	20.0	174,951	3.2	27,932
Montana	299,898	90.9	272,508	8.2	24,654	.9	2,736
Nebraska	410,807	84.2	345,849	14.0	57,528	1.8	7,430
Nevada	75,497	81.9	61,802	12.2	9,229	5.9	4,466
New Hampshire	146,501	66.5	97,434	24.4	35,789	9.1	13,278
New Mexico	276,166	80.9	223,381	14.0	38,627	5.1	14,158
North Carolina	838,939	77.1	646,529	17.2	144,516	5.7	47,894
North Dakota	142,025	88.8	126,137	9.3	13,250	1.9	2,638

TABLE 14.—*Vehicle-mileage of trucks and combinations which traveled intrastate, interstate, and transstate on ALL RURAL ROADS in each State in the year 1940*—Continued

State	Total truck travel	Intrastate		Interstate		Transstate	
	Thousands of vehicle-miles	*Per-cent*	*Thousands of vehicle-miles*	*Per-cent*	*Thousands of vehicle-miles*	*Per-cent*	*Thousands of vehicle-miles*
Ohio	1,365,712	77.7	1,061,515	18.0	245,091	4.3	59,106
Oklahoma	694,743	79.2	550,289	18.9	131,019	1.9	13,435
Oregon	329,529	91.1	300,219	7.9	26,011	1.0	3,299
Pennsylvania	1,421,912	82.0	1,165,947	15.2	216,280	2.8	39,685
Rhode Island	64,243	65.4	42,033	29.0	18,593	5.6	3,617
South Carolina	481,960	70.4	339,563	19.5	93,841	10.1	48,556
South Dakota	247,320	82.6	204,365	16.8	41,434	.6	1,521
Tennessee	612,204	83.9	513,635	13.2	80,891	2.9	17,678
Texas	1,980,100	93.9	1,858,928	5.7	114,050	.4	7,122
Utah	159,653	91.3	145,746	7.8	12,468	.9	1,439
Vermont	108,350	80.0	86,720	17.9	19,411	2.1	2,219
Virginia	752,422	70.4	529,522	21.0	158,243	8.6	64,657
Washington	493,280	90.7	447,190	8.7	42,955	.6	3,135
West Virginia	390,831	82.6	322,926	14.4	56,201	3.0	11,704
Wisconsin	797,386	87.6	698,096	10.6	84,692	1.8	14,598
Wyoming	150,627	70.7	106,541	20.0	30,160	9.3	13,926
Subtotal	25,445,762	82.8	21,076,298	14.1	3,581,089	3.1	788,375
Other States [1]	4,404,539	75.9	3,343,647	19.7	866,990	4.4	193,902
Totals and averages	29,850,301	81.8	24,419,945	14.9	4,418,079	3.3	982,277

[1] Includes Arkansas, Connecticut, Delaware, Georgia, Maine, New Jersey, and New York.

TABLE 15.—*Percentage loaded, average weight of laad and ton-mileage of load carried by trucks and combinations intrastate, interstate, and transstate an MAIN RURAL ROADS in each State in the year 1940*

State	Intrastate				Interstate				Transstate				Total ton-miles
	Percentage loaded	Carried load			Percentage loaded	Carried load			Percentage loaded	Carried load			
		Average weight	Ton-miles	Percentage of State total		Average weight	Ton-miles	Percentage of State total		Average weight	Ton-miles	Percentage of State total	
		Tons	*Thousands*			*Tons*	*Thousands*			*Tons*	*Thousands*		*Thousands*
Alabama	56.2	2.63	338,939	66.1	68.6	3.71	155,584	30.3	70.4	3.36	18,361	3.6	512,884
Arizona	55.4	2.54	117,513	36.6	82.5	8.26	164,674	51.4	75.0	6.52	38,433	12.0	320,620
California	71.7	5.25	3,948,330	93.1	87.0	8.53	291,136	6.8	79.8	2.06	1,791	.1	4,241,257
Colorado	66.5	2.39	458,967	66.3	76.0	4.06	230,237	33.3	92.6	1.49	3,019	.4	692,223
Florida	47.6	3.60	645,993	74.3	59.1	4.78	220,928	25.4	69.7	3.75	2,389	.3	869,310
Idaho	58.3	1.96	161,838	72.1	74.8	3.31	57,930	25.8	85.8	1.80	4,678	2.1	224,446
Illinois	60.7	3.04	1,053,649	48.7	76.2	5.01	842,889	39.0	78.8	6.05	266,809	12.3	2,163,347
Indiana	67.4	2.69	910,169	38.6	73.2	5.57	780,499	33.1	85.8	7.48	665,495	28.3	2,356,163
Iowa	66.9	2.47	580,470	56.5	80.5	4.08	311,165	30.3	90.7	6.39	135,861	13.2	1,027,496
Kansas	58.8	3.10	467,984	51.1	65.5	4.59	382,008	41.7	78.4	3.79	66,354	7.2	916,346
Kentucky	62.5	2.15	391,986	48.9	80.3	3.55	229,122	28.6	90.1	5.56	179,787	22.5	800,895
Louisiana	55.9	2.71	351,726	74.2	72.0	3.08	111,809	23.6	71.5	3.72	10,655	2.2	474,190
Maryland	65.0	2.93	392,661	35.8	70.7	3.93	481,734	43.9	77.1	5.82	223,377	20.3	1,097,772
Massachusetts	70.4	2.49	444,594	51.5	82.6	5.80	386,528	44.8	83.5	5.93	31,963	3.7	863,085
Michigan	66.2	3.08	1,351,295	75.0	88.3	5.13	448,804	24.9	77.7	2.74	650	.1	1,800,749
Minnesota	69.5	2.73	692,452	78.2	71.5	3.96	180,875	20.4	77.9	2.72	11,946	1.4	885,273
Mississippi	57.6	2.21	520,642	68.6	71.3	2.51	173,477	22.9	78.5	3.03	64,612	8.5	758,731
Missouri	59.2	2.59	764,324	53.0	71.8	4.71	565,774	39.3	67.5	5.95	111,350	7.7	1,441,448
Montana	19.3	3.06	238,177	70.7	64.0	6.06	85,524	25.4	85.0	5.64	13,097	3.9	336,798
Nebraska	67.3	2.37	311,114	63.6	71.7	3.96	148,815	30.4	81.8	5.08	29,578	6.0	489,507
Nevada	57.9	2.90	62,094	52.0	77.1	4.27	25,999	21.8	92.0	7.67	31,340	26.2	119,433
New Hampshire	67.4	2.19	84,050	42.4	81.2	3.07	70,445	35.5	87.4	4.09	43,948	22.1	198,443
New Mexico	54.3	2.43	149,407	51.1	72.8	3.43	90,475	30.9	85.4	4.39	52,737	18.0	292,619
North Carolina	62.9	3.63	1,125,740	64.6	74.5	4.27	441,285	25.3	72.1	5.46	175,746	10.1	1,742,771
North Dakota	65.3	3.13	166,154	79.7	76.3	3.44	31,968	15.3	91.1	4.42	10,436	5.0	208,558
Ohio	57.4	3.85	1,541,220	47.9	73.6	7.31	1,255,221	39.1	82.2	8.73	418,676	13.0	3,215,117
Oklahoma	57.3	2.96	601,864	62.3	65.5	4.04	322,818	33.4	70.5	4.39	40,926	4.3	965,608
Pennsylvania	65.4	2.75	1,476,958	66.2	71.0	4.25	616,297	27.6	78.6	4.48	138,545	6.2	2,231,800
Rhode Island	61.7	1.80	35,174	37.6	70.9	3.52	44,717	47.8	76.1	5.28	13,641	14.6	93,532
South Carolina	62.9	3.48	516,346	53.3	70.5	4.44	280,528	29.0	72.6	4.89	171,204	17.7	968,078
South Dakota	61.9	1.94	127,808	61.3	72.4	3.19	78,111	37.4	78.5	2.48	2,750	1.3	208,669
Tennessee	62.4	2.39	437,474	64.3	71.2	3.65	189,701	27.9	75.5	4.11	53,425	7.8	680,600
Texas	60.2	2.22	1,751,276	87.8	67.8	3.21	232,361	11.7	79.9	1.87	10,672	.5	1,994,309
Utah	62.0	2.79	169,395	81.0	79.5	4.03	36,222	17.3	92.2	2.68	3,549	1.7	209,166
Virginia	61.4	2.69	608,490	44.5	73.8	4.61	511,693	37.5	82.3	4.67	246,377	18.0	1,366,560
Washington	59.1	3.31	596,687	77.3	74.7	5.40	161,681	20.9	72.0	6.00	13,542	1.8	771,910
West Virginia	54.5	2.16	222,440	55.7	82.9	3.52	139,194	34.9	77.9	4.35	37,730	9.4	399,364
Wisconsin	71.7	2.59	746,185	65.5	87.1	4.00	269,594	23.6	93.1	9.34	124,282	10.9	1,140,061
Wyoming	52.9	3.78	128,230	50.8	67.1	4.67	87,445	34.7	63.2	4.22	36,667	14.5	252,342
Subtotal	62.5	3.00	24,689,815	62.8	73.6	4.61	11,135,267	28.3	80.0	5.70	3,506,398	8.9	39,331,480
Other States [1]	62.4	2.81	4,024,558	54.7	71.4	4.38	2,628,930	35.8	75.6	4.72	700,753	9.5	7,354,241
Totals and averages	62.5	2.97	28,714,373	61.5	73.1	4.57	13,764,197	29.5	79.1	5.51	4,207,151	9.0	46,685,721

[1] Includes Arkansas, Connecticut, Delaware, Georgia, Maine, New Jersey, New York, Oregon, and Vermont.

TABLE 16.—*Percentage loaded, average weight of load, and ton-mileage of load carried by trucks and combinations intrastate, interstate, and transstate on LOCAL RURAL ROADS in each State in the year 1940*

State	Intrastate				Interstate				Transstate				Total ton-miles
	Percentage loaded	Carried load			Percentage loaded	Carried load			Percentage loaded	Carried load			
		Average weight	Ton-miles	Percentage of State total		Average weight	Ton-miles	Percentage of State total		Average weight	Ton-miles	Percentage of State total	
		Tons	*Thousands*			*Tons*	*Thousands*			*Tons*	*Thousands*		*Thousands*
Idaho	59.7	1.60	82,600	87.2	76.6	2.70	11,855	12.5	88.1	1.47	239	0.3	94,694
Indiana	66.6	1.68	423,092	93.4	72.3	3.47	28,446	6.3	84.8	4.67	1,545	.3	453,083
Iowa	69.7	1.63	262,538	93.3	83.8	2.70	17,827	6.3	94.4	4.23	955	.4	281,320
Michigan	67.5	1.50	321,123	94.5	90.1	2.50	18,313	5.4	79.3	1.33	344	.1	339,780
Minnesota	68.7	1.93	299,883	96.4	70.7	2.80	11,017	3.5	77.0	1.92	344	.1	311,244
Nebraska	68.2	1.76	181,859	93.8	72.7	2.95	11,069	5.7	82.9	3.78	983	.5	193,911
Nevada	61.1	2.13	32,296	90.2	80.5	3.13	3,366	9.4	92.8	5.64	141	.4	35,803
New Hampshire	71.3	2.06	59,525	72.9	85.9	2.89	18,600	22.8	92.5	3.86	3,489	4.3	81,614
New Mexico	60.0	2.07	136,842	95.8	80.5	2.92	5,559	3.9	94.4	3.73	400	.3	142,801
Ohio	58.5	2.85	607,966	92.2	75.0	5.42	47,450	7.2	83.8	6.47	4,084	.6	659,500
South Carolina	65.9	2.14	145,936	94.2	73.8	2.73	8,270	5.3	76.0	3.00	739	.5	154,915
South Dakota	63.8	1.73	108,069	86.9	74.6	2.84	16,141	13.0	80.2	2.21	188	.1	124,401
Tennessee	63.5	1.92	269,910	93.8	72.5	2.93	16,588	5.8	77.0	3.31	1,169	.4	287,667
Utah	63.3	2.00	60,341	95.6	81.1	2.88	2,749	4.4					63,090
West Virginia	58.6	1.81	142,274	85.7	89.1	2.95	22,231	13.4	83.7	3.27	1,574	.9	166,079
Wisconsin	72.9	1.66	358,576	95.1	88.6	2.70	17,473	4.6	94.7	3.34	961	.3	377,010
Wyoming	55.5	2.66	62,550	91.9	70.3	3.28	5,182	7.6	66.3	2.97	353	.5	68,085
Subtotal	65.6	1.89	3,555,380	92.6	79.6	3.16	262,139	6.9	85.3	3.80	17,508	.5	3,835,027
Other States [1]	63.4	2.06	7,938,975	93.0	75.0	3.38	565,409	6.6	78.8	3.72	33,212	.4	8,537,596
Totals and averages	63.9	2.01	11,494,355	92.9	76.4	3.31	827,548	6.7	80.8	3.74	50,720	.4	12,372,623

[1] Includes Alabama, Arizona, Arkansas, California, Colorado, Connecticut, Delaware, Florida, Georgia, Illinois, Kansas, Kentucky, Louisiana, Maine, Maryland, Massachusetts, Mississippi, Missouri, Montana, New Jersey, New York, North Carolina, North Dakota, Oklahoma, Oregon, Pennsylvania, Rhode Island, Texas, Vermont, Virginia, and Washington.

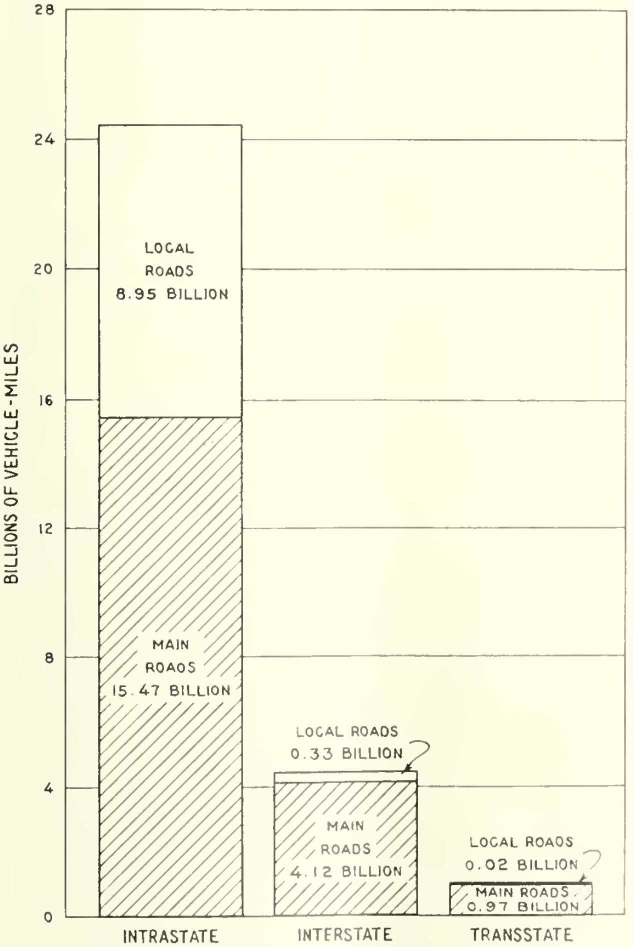

FIGURE 11.—VEHICLE-MILES OF TRUCKS TRAVELING INTRASTATE, INTERSTATE AND TRANSSTATE ON MAIN AND LOCAL RURAL ROADS IN THE YEAR 1940.

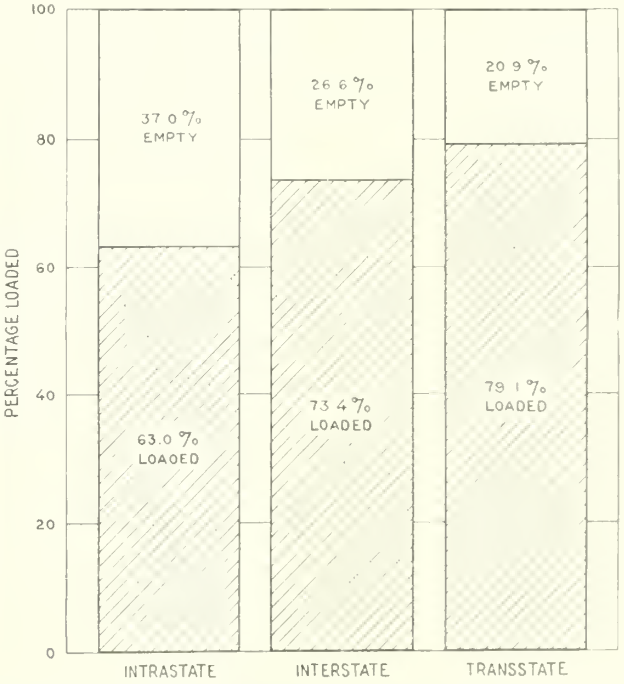

FIGURE 12.—LOADED AND EMPTY TRUCKS AS PERCENTAGES OF TRUCKS TRAVELING INTRASTATE, INTERSTATE, AND TRANSSTATE ON ALL RURAL ROADS IN THE YEAR 1940.

alone. The two classifications combined accounted for 38.5 percent of the ton-miles of hauling on main roads, and for 31.9 percent of that on all rural roads, as shown in tables 15 and 17. A greater proportion of the trucks engaged in hauling between points in different States were loaded and they carried heavier loads, on the average, than those on intrastate trips. This is brought out clearly in figures 12 and 13.

TABLE 17.—*Percentage loaded, average weight of load, and ton-mileage of load carried by trucks and combinations intrastate, interstate, and transstate on ALL RURAL ROADS in each State in the year 1940*

State	Intrastate				Interstate				Transstate				Total ton-miles
	Percentage loaded	Carried load			Percentage loaded	Carried load			Percentage loaded	Carried load			
		Average weight	Ton-miles	Percentage of State total		Average weight	Ton-miles	Percentage of State total		Average weight	Ton-miles	Percentage of State total	
		Tons	*Thousands*			*Tons*	*Thousands*			*Tons*	*Thousands*		*Thousands*
Alabama	57.4	2.38	677,378	77.5	69.0	3.61	177,368	20.3	70.5	3.34	18,926	2.2	873,672
Arizona	57.3	2.41	227,365	50.3	83.2	8.16	185,163	41.0	75.1	6.50	39,223	8.7	451,751
California	71.0	4.23	5,109,097	94.4	86.8	8.14	304,949	5.6	79.8	2.06	1,791	[1]	5,415,837
Colorado	66.9	2.30	572,746	70.3			238,487	29.3	92.6	1.49	3,019	.4	814,252
Florida	47.6	3.24	841,859	78.3	59.1	4.68	230,916	21.5	69.7	3.74	2,389	.2	1,075,164
Idaho	58.8	1.82	244,438	76.6	75.1	3.19	69,785	21.9	86.0	1.78	4,917	1.5	319,140
Illinois	61.8	2.72	1,524,689	56.8	76.5	4.47	889,975	33.2	78.8	6.03	269,368	10.0	2,684,032
Indiana	67.1	2.26	1,333,261	47.5	73.2	5.46	808,945	28.8	85.8	7.47	667,040	23.7	2,809,246
Iowa	68.0	2.13	843,008	64.4	80.7	3.97	328,992	25.1	90.7	6.36	136,816	10.5	1,308,816
Kansas	59.8	2.37	732,275	60.6	65.7	4.36	407,922	33.8	78.4	3.75	67,267	5.6	1,207,464
Kentucky	64.0	2.15	586,379	57.9	80.6	3.55	244,125	24.1	90.1	5.56	182,048	18.0	1,012,552
Louisiana	56.8	2.41	530,969	80.2	72.3	3.00	120,204	18.2	71.5	3.72	10,655	1.6	661,828
Maryland	65.7	2.89	517,975	41.8	70.8	3.92	495,980	40.1	77.1	5.81	224,656	18.1	1,238,611
Massachusetts	71.6	2.31	932,559	66.1	83.0	5.69	445,055	31.5	83.6	5.89	33,402	2.4	1,411,016
Michigan	66.6	2.56	1,672,418	78.1	88.5	4.92	467,117	21.8	78.4	2.01	994	.1	2,140,529
Minnesota	69.2	2.43	992,335	82.9	71.5	3.97	191,892	16.1	77.8	2.69	12,290	1.0	1,196,517
Mississippi	58.6	1.97	737,579	74.8	71.6	2.45	183,580	18.6	78.6	3.01	65,453	6.6	986,612
Missouri	59.4	2.45	976,372	58.3	71.8	4.67	586,810	35.0	67.5	5.94	111,934	6.7	1,675,116
Montana	49.7	2.77	374,828	77.9	64.1	5.91	93,425	19.4	84.9	5.64	13,097	2.7	481,350
Nebraska	67.7	2.10	492,973	72.1	71.8	3.87	159,884	23.4	81.8	5.03	30,561	4.5	683,418
Nevada	59.2	2.58	94,390	60.8	77.6	4.10	29,365	18.9	92.1	7.66	31,481	20.3	155,236
New Hampshire	69.0	2.13	143,575	51.3	82.2	3.03	89,045	31.8	87.8	4.07	47,437	16.9	280,057
New Mexico	57.1	2.24	286,249	65.7	75.9	3.28	96,034	22.1	85.5	4.39	53,137	12.2	435,420
North Carolina	63.1	3.27	1,333,095	67.7	74.6	4.20	452,302	23.0	72.1	5.36	183,322	9.3	1,968,719
North Dakota	65.5	2.84	234,960	84.0	76.3	3.37	34,050	12.2	91.1	4.10	10,574	3.8	279,584
Ohio	57.8	3.50	2,149,186	55.5	73.6	7.22	1,302,671	33.6	82.2	8.70	422,760	10.9	3,874,617
Oklahoma	56.9	2.54	796,551	67.8	65.4	3.93	336,811	28.7	70.4	4.36	41,299	3.5	1,174,661
Pennsylvania	65.7	2.55	1,953,180	71.4	71.1	1.19	644,448	23.5	58.7	4.47	139,511	5.1	2,737,139
Rhode Island	62.4	1.78	46,727	43.4	71.0	3.51	46,411	43.1	76.3	5.27	14,548	13.5	107,686
South Carolina	63.8	3.06	662,282	59.0	70.6	4.36	288,798	25.7	72.6	4.87	171,943	15.3	1,123,023
South Dakota	62.8	1.84	235,877	70.8	72.8	3.12	94,255	28.3	78.6	2.16	2,938	.9	333,070
Tennessee	63.0	2.19	707,384	73.1	71.3	3.58	206,289	21.3	75.6	4.09	54,594	5.6	968,267
Texas	60.1	2.02	2,254,332	89.9	67.8	3.15	243,151	9.7	79.9	1.87	10,672	.4	2,508,155
Utah	62.4	2.53	229,736	84.4	79.7	3.92	38,971	14.3	92.1	2.68	3,549	1.3	272,256
Virginia	62.6	2.51	830,867	51.5	71.0	4.56	533,850	33.1	82.3	4.66	248,010	15.4	1,612,727
Washington	59.3	2.96	785,652	81.1	74.8	5.28	169,636	17.5	72.0	6.00	13,542	1.4	968,830
West Virginia	56.2	2.01	364,714	64.5	83.9	3.43	161,425	28.5	78.2	4.29	39,304	7.0	565,443
Wisconsin	72.2	2.19	1,104,761	72.8	87.3	3.88	287,067	18.9	93.1	9.21	125,243	8.3	1,517,071
Wyoming	53.9	3.32	190,780	59.5	67.3	4.56	92,627	28.9	63.2	5.20	37,020	11.6	320,427
Subtotal	63.0	2.64	34,324,801	69.1	58.5	5.69	11,777,780	23.7	78.6	5.67	3,546,730	7.2	49,649,311
Other States [2]	63.2	2.49	5,883,927	62.5	71.7	4.31	2,813,965	29.9	81.2	4.71	711,141	7.6	9,409,033
Totals and averages	63.0	2.61	40,208,728	68.1	73.4	4.47	14,591,745	24.7	79.1	5.48	4,257,871	7.2	59,058,344

[1] Less than 0.05 percent.

[2] Includes Arkansas, Connecticut, Delaware, Georgia, Maine, New Jersey, New York, Oregon, and Vermont.

EXTENT OF TRIPS INDICATED BY NUMBER OF COUNTIES

In 17 States, studies were made on the basis of the number of counties into which a trip extended, and table 18 is based on these studies. County size varies considerably in different sections of the country and to aid in interpreting the table in terms of trip length, the average county area is shown for each of the States. In Nevada, in which counties average 6,458 square miles in area, or about 80 miles square, a trip within one county might be long compared to a trip within a county in Indiana where counties average only 394 square miles, or about 20 miles square. However, towns and railroads are so much farther apart in the west that longer distances must be traveled to get to town, or to a railroad station, and the average trip within a county would still be classed as relatively short in States like Nevada.

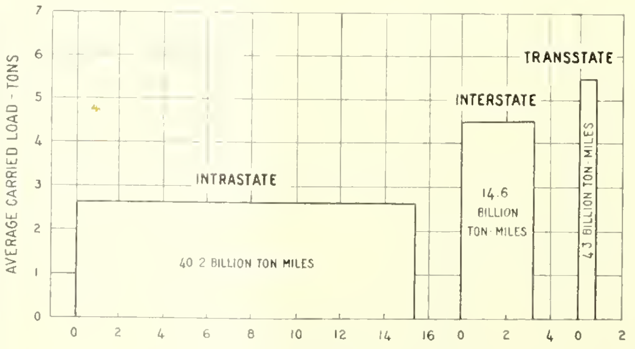

FIGURE 13.—TON-MILES OF LOAD CARRIED BY TRUCKS TRAVELING INTRASTATE, INTERSTATE, AND TRANSSTATE ON ALL RURAL ROADS IN THE YEAR 1940.

TABLE 18.—*Vehicle-mileage traveled by trucks and combinations making trips extending into one county only, two counties, and three or more counties on MAIN AND ON LOCAL RURAL ROADS in each of 17 States*

State	Total truck travel	Trips requiring travel in— One county		Two counties		Three or more counties		Average area of a county
	Thousands of vehicle-miles	*Percent*	*Thousands of vehicle-miles*	*Percent*	*Thousands of vehicle-miles*	*Percent*	*Thousands of vehicle-miles*	*Square miles*
MAIN RURAL ROADS								
Idaho	168, 295	46. 6	78, 125	22. 7	38, 203	30. 7	51, 667	1, 882
Indiana	797, 144	20. 6	164, 212	20. 9	166, 603	58. 5	466, 329	394
Iowa	469, 331	24. 6	115, 456	21. 7	101, 845	53. 7	252, 030	566
Michigan	763, 041	28. 6	218, 230	38. 6	294, 534	32. 8	250, 277	687
Minnesota	433, 872	42. 0	182, 227	29. 2	126, 690	28. 8	124, 955	920
Nebraska	254, 168	20. 9	53, 223	17. 8	45, 166	61. 3	155, 779	824
Nevada	49, 341	52. 9	26, 101	30. 0	14, 802	17. 1	8, 438	6, 458
New Hampshire	97, 620	32. 2	31, 434	22. 4	21, 867	45. 4	44, 319	902
New Mexico	163, 318	44. 6	72, 840	16. 7	27, 274	38. 7	63, 204	3, 920
Ohio	989, 042	22. 7	224, 513	21. 7	214, 622	55. 6	549, 907	467
Oregon	235, 643	48. 6	114, 522	28. 7	67, 630	22. 7	53, 491	2, 676
South Carolina	373, 895	23. 0	85, 996	32. 9	123, 011	44. 1	164, 888	665
South Dakota	141, 482	31. 0	43, 859	23. 5	33, 249	45. 5	64, 374	1, 109
Tennessee	382, 650	25. 1	96, 045	24. 8	94, 897	50. 1	191, 708	763
Utah	110, 714	36. 5	40, 411	27. 4	30, 335	36. 1	39, 968	2, 840
West Virginia	247, 328	37. 6	92, 995	28. 6	70, 736	33. 8	83, 597	438
Wisconsin	492, 893	31. 6	155, 656	23. 5	115, 780	44. 9	221, 457	771
Totals and averages	6, 169, 777	29. 1	1, 796, 145	25. 7	1, 587, 244	45. 2	2, 786, 388	1, 071
LOCAL RURAL ROADS								
Idaho	92, 570	76. 3	70, 631	16. 9	15, 644	6. 8	6, 295	1, 882
Indiana	390, 438	65. 6	256, 127	27. 2	106, 200	7. 2	28, 111	394
Iowa	238, 765	65. 3	155, 913	23. 9	57, 065	10. 8	25, 787	566
Michigan	325, 750	68. 1	221, 836	24. 4	79, 483	7. 5	24, 431	687
Minnesota	232, 199	72. 9	169, 273	23. 4	54, 334	3. 7	8, 592	920
Nebraska	156, 639	68. 5	107, 376	19. 9	31, 171	11. 6	18, 092	824
Nevada	26, 156	88. 6	23, 174	7. 8	2, 040	3. 6	942	6, 458
New Hampshire	48, 881	65. 0	31, 773	20. 6	10, 069	14. 4	7, 039	902
New Mexico	112, 848	81. 6	92, 084	16. 2	18, 284	2. 2	2, 483	3, 920
Ohio	376, 670	64. 5	242, 952	26. 8	100, 948	8. 7	32, 770	467
Oregon	93, 886	79. 9	75, 015	17. 1	16, 054	3. 0	2, 817	2, 676
South Carolina	108, 065	72. 8	78, 671	22. 1	23, 883	5. 1	5, 511	665
South Dakota	105, 838	67. 3	71, 229	20. 1	21, 273	12. 6	13, 336	1, 109
Tennessee	229, 554	68. 7	157, 704	21. 7	49, 813	9. 6	22, 037	763
Utah	48, 939	80. 6	39, 445	14. 6	7, 145	4. 8	2, 349	2, 840
West Virginia	143, 503	72. 0	103, 322	21. 8	31, 283	6. 2	8, 898	438
Wisconsin	304, 493	69. 7	212, 232	24. 4	74, 296	5. 9	17, 965	771
Totals and averages	3, 035, 194	69. 5	2, 108, 757	23. 0	698, 982	7. 5	227, 455	1, 071
ALL RURAL ROADS								
Idaho	260, 865	57. 1	149, 056	20. 7	53, 847	22. 2	57, 962	1, 882
Indiana	1, 187, 582	35. 4	420, 339	23. 0	272, 803	41. 6	494, 440	394
Iowa	708, 096	38. 3	271, 369	22. 5	158, 910	39. 2	277, 817	566
Michigan	1, 088, 791	40. 4	440, 066	34. 4	374, 017	25. 2	274, 708	687
Minnesota	666, 071	52. 8	354, 500	27. 2	181, 024	20. 0	133, 547	920
Nebraska	410, 807	39. 1	160, 599	18. 6	76, 337	42. 3	173, 871	824
Nevada	75, 497	65. 3	49, 275	22. 3	16, 842	12. 4	9, 380	6, 458
New Hampshire	146, 501	43. 1	63, 207	21. 9	31, 936	35. 0	51, 358	902
New Mexico	276, 166	59. 7	164, 924	16. 5	45, 555	23. 8	65, 687	3, 920
Ohio	1, 365, 712	34. 2	467, 465	23. 1	315, 570	42. 7	582, 677	467
Oregon	329, 529	57. 5	189, 537	25. 1	83, 684	17. 4	56, 308	2, 676
South Carolina	481, 960	34. 2	164, 667	30. 5	146, 894	35. 3	170, 399	665
South Dakota	247, 320	46. 5	115, 088	22. 1	54, 522	31. 4	77, 710	1, 109
Tennessee	612, 204	41. 5	253, 749	23. 6	144, 710	34. 9	213, 745	763
Utah	159, 653	50. 0	79, 856	23. 5	37, 480	26. 5	42, 317	2, 840
West Virginia	390, 831	50. 2	196, 317	26. 1	102, 019	23. 7	92, 495	438
Wisconsin	797, 386	46. 2	367, 888	23. 8	190, 076	30. 0	239, 422	771
Totals and averages	9, 204, 971	42. 4	3, 904, 902	24. 8	2, 286, 226	32. 8	3, 013, 843	1, 071

It is difficult to interpret data on trips requiring travel in two counties in terms of trip length because they may be very short trips from farms in one county to a town just across the county line, or they may extend almost across the two counties. The most that can be said about them is that they cannot be longer than the distance across two counties. Most of the trips requiring travel in three or more counties are probably relatively long, though there are exceptional cases where short trips cut across a corner or narrow neck of a county to a third county.

Interpreting figure 14 in such terms, it may be said that, on all rural roads in the 17 States in which a study was made of trip extent in terms of numbers of counties traversed, about 42 percent of the truck-mileage was on short trips, about 33 percent was on long trips, and the remaining 25 percent was in part on short trips and in part on moderately long trips. On main roads, corresponding percentages would be 29 percent, 45 percent, and 26 percent for the short, long, and short or moderately long classifications, respectively. Such interpretations are perhaps more en-

TABLE 19.—*Percentage of vehicles loaded, average weight of load, and ton-mileage of load carried by trucks and combinations on trips requiring travel in one county only, two counties, and three or more counties on MAIN RURAL ROADS in each of 16 States*

State	Trips requiring travel in—												Total ton-miles
	One county				Two counties				Three or more counties				
	Percentage loaded	Carried load			Percentage loaded	Carried load			Percentage loaded	Carried load			
		Average weight	Ton-miles	Percentage of State total		Average weight	Ton-miles	Percentage of State total		Average weight	Ton-miles	Percentage of State total	
		Tons	*Thousands*			*Tons*	*Thousands*			*Tons*	*Thousands*		*Thousands*
Idaho	53.8	1.32	55,834	24.9	63.5	2.50	60,664	27.0	70.5	2.96	107,948	48.1	224,446
Indiana	66.0	1.40	152,307	6.5	67.7	2.82	317,948	13.5	71.3	5.44	1,885,908	80.0	2,356,163
Iowa	62.2	1.99	142,990	13.9	66.8	2.41	165,860	16.2	76.5	3.73	718,646	69.9	1,027,496
Michigan	51.6	1.59	179,450	10.0	68.6	3.06	618,026	31.3	84.7	4.73	1,003,273	55.7	1,800,749
Minnesota	69.0	2.14	269,364	30.4	70.1	3.21	284,621	32.2	71.3	3.72	331,288	37.4	885,273
Nebraska	66.1	1.80	63,130	12.9	67.0	2.22	67,084	13.7	70.0	3.29	359,293	73.4	489,507
Nevada	53.3	2.79	38,796	32.5	72.5	3.44	36,942	30.9	82.7	6.26	43,695	36.6	119,433
New Hampshire	63.0	2.04	40,491	20.4	73.1	2.37	37,919	19.1	82.1	3.30	120,033	60.5	198,443
New Mexico	49.9	2.21	80,234	27.4	61.6	2.73	45,913	15.7	73.8	3.57	166,472	56.9	292,619
Ohio	50.0	2.16	242,649	7.5	57.8	3.83	474,902	14.8	69.7	6.51	2,497,566	77.7	3,215,117
South Carolina	59.5	3.01	154,126	15.9	66.3	3.93	320,352	33.1	69.1	4.33	493,600	51.0	968,078
South Dakota	78.3	1.22	31,165	14.9	63.7	2.23	47,191	22.6	69.3	2.92	130,310	62.5	208,669
Tennessee	58.5	1.71	97,931	14.4	62.6	2.38	111,546	20.8	69.0	3.34	441,123	61.8	680,600
Utah	55.0	1.99	44,164	21.1	64.1	2.91	56,613	27.1	73.6	3.69	108,389	51.8	209,166
West Virginia	40.4	1.57	59,032	14.8	66.3	2.46	115,273	28.8	79.6	3.38	225,059	56.1	399,361
Wisconsin	63.7	1.82	180,481	15.8	72.6	2.56	215,561	18.9	83.7	4.02	744,019	65.3	1,140,061
Totals and averages	57.7	1.89	1,832,144	12.9	66.2	2.99	3,006,418	21.1	74.3	4.62	9,376,622	66.0	14,215,184

TABLE 20.—*Percentage of vehicles loaded, average weight of load, and ton-mileage of load carried by trucks and combinations on trips requiring travel in one county only, two counties, and three or more counties on LOCAL RURAL ROADS in each of 16 States*

State	Trips requiring travel in—												Total ton-miles
	One county				Two counties				Three or more counties				
	Percentage loaded	Carried load			Percentage loaded	Carried load			Percentage loaded	Carried load			
		Average weight	Ton-miles	Percentage of State total		Average weight	Ton-miles	Percentage of State total		Average weight	Ton-miles	Percentage of State total	
		Tons	*Thousands*			*Tons*	*Thousands*			*Tons*	*Thousands*		*Thousands*
Idaho	58.2	1.40	57,559	60.8	67.2	2.12	25,442	26.9	73.5	2.53	11,693	12.3	94,694
Indiana	65.0	1.38	230,426	50.9	69.6	2.25	166,053	36.6	71.8	2.80	56,594	12.5	453,083
Iowa	67.3	1.41	147,539	52.5	71.3	2.03	85,857	30.5	78.1	2.38	47,924	17.0	281,320
Michigan	60.6	1.18	158,650	46.7	82.5	1.97	128,932	37.9	89.0	2.40	52,198	15.1	339,780
Minnesota	68.4	1.70	196,441	63.1	69.6	2.59	97,878	31.5	70.3	2.80	16,925	5.4	311,244
Nebraska	67.8	1.54	111,962	57.7	69.2	2.27	48,890	25.2	70.7	2.58	33,059	17.1	193,911
Nevada	60.2	2.08	28,955	80.9	76.9	2.83	4,435	12.4	80.9	3.17	2,413	6.7	35,803
New Hampshire	69.0	1.96	42,883	52.5	80.9	2.50	20,359	25.0	86.6	3.01	18,372	22.5	81,614
New Mexico	59.9	1.97	108,566	76.0	63.4	2.58	29,898	20.9	62.2	2.81	4,337	3.1	142,801
Ohio	74.1	2.11	277,900	42.1	65.6	3.96	265,605	40.1	72.0	4.87	114,995	17.5	659,500
South Carolina	61.6	2.03	103,383	66.7	70.0	2.46	41,068	26.5	72.0	2.64	10,494	6.8	154,945
South Dakota	61.5	1.38	60,734	48.8	69.9	2.48	36,878	29.7	73.0	2.75	26,789	21.5	124,401
Tennessee	61.3	1.66	160,213	55.7	68.6	2.45	83,823	29.1	71.3	2.78	43,631	15.2	287,667
Utah	60.5	1.83	43,719	69.3	76.4	2.61	14,221	22.5	79.0	2.77	5,117	8.2	63,090
West Virginia	51.7	1.53	81,459	49.0	81.7	2.44	62,416	37.6	87.8	2.84	22,174	13.4	166,079
Wisconsin	70.1	1.54	229,088	60.8	80.0	1.92	114,185	30.3	84.3	2.22	33,737	8.9	377,010
Totals and averages	62.7	1.60	2,039,477	51.1	72.7	2.47	1,226,983	32.6	76.5	2.91	500,482	13.3	3,766,942

lightening when made on a State-by-State rather than on a national basis. The estimates for individual States are shown in table 18.

Separate values for percentage of trucks loaded and average carried load, classed according to number of counties in which the travel occurred were not directly available, because such basis of classifications was not used at the stations at which trucks were weighed. However, tables 15, 16, and 17 show that both the percentage of trucks loaded and the average carried load were greatest for transstate trips, next greatest for interstate trips, and least for intrastate trips. It is reasonable to assume that, within each of these classifications, the percentage of trucks loaded and the average carried load increased as the extent of trip, in terms of the number of counties in which the travel occurred, increased. Values shown in tables 19, 20, and 21 were estimated from curves plotted on the basis of such an assumption, as explained in the appendix. Figure 15 shows the percentage distribution of ton-mileage according to number of counties in which the travel occurred for main and local roads, separately and combined. Naturally the long-haul classification contains a higher percentage of the ton-mileage than of the vehicle-mileage, because a greater percentage of the trucks traveling long distances are loaded and they weigh more, on the average, than those traveling short distances.

TABLE 21.—*Percentage of vehicles loaded, average weight of load, and ton-mileage of load carried by trucks and combinations on trips requiring travel in one county only, two counties, and three or more counties on ALL RURAL ROADS in each of 16 States*

State	Trips requiring travel in												
	One county				Two counties				Three or more counties				Total ton-miles
	Percentage loaded	Carried load			Percentage loaded	Carried load			Percentage loaded	Carried load			
		Average weight	Ton-miles	Percentage of State total		Average weight	Ton miles	Percentage of State total		Average weight	Ton-miles	Percentage of State total	
		Tons	*Thousands*			*Tons*	*Thousands*			*Tons*	*Thousands*		*Thousands*
Idaho	55.9	1.36	113,393	35.5	64.6	2.48	86,106	27.0	70.8	2.91	119,641	37.5	319,140
Indiana	65.4	1.39	382,733	13.6	68.4	2.59	484,011	17.2	74.2	5.30	1,942,502	69.2	2,809,246
Iowa	65.1	1.64	290,529	22.2	69.5	2.28	251,717	19.2	76.6	3.60	766,570	58.6	1,308,816
Michigan	56.2	1.37	338,100	15.8	71.6	2.79	716,958	34.9	85.1	4.51	1,055,171	49.3	2,110,229
Minnesota	68.7	1.93	465,805	38.9	69.9	3.02	382,499	32.0	71.2	3.66	348,213	29.1	1,196,517
Nebraska	67.2	1.62	175,092	25.6	67.9	2.24	115,974	17.0	70.1	3.22	392,352	57.1	683,418
Nevada	56.5	2.43	67,751	43.6	73.0	3.36	41,377	26.7	82.5	5.96	46,108	29.7	155,236
New Hampshire	66.0	2.00	83,374	29.8	75.5	2.42	58,278	20.8	82.7	3.26	138,405	49.4	280,057
New Mexico	55.4	2.06	188,800	43.4	62.3	2.66	75,811	17.4	73.4	3.54	170,809	39.2	435,420
Ohio	52.1	2.14	520,549	13.4	60.7	3.87	744,507	19.7	69.9	6.42	2,612,564	67.4	3,877,620
South Carolina	61.9	2.53	257,509	22.9	65.9	3.67	361,420	32.2	69.2	4.27	504,094	44.9	1,123,023
South Dakota	60.3	1.33	91,899	27.6	66.1	2.33	84,072	25.2	70.0	2.89	157,099	47.2	333,070
Tennessee	60.2	1.69	258,144	26.7	64.7	2.41	225,369	23.3	69.2	3.28	484,754	50.0	968,267
Utah	57.7	1.91	87,883	32.3	66.4	2.85	70,837	26.0	73.8	3.63	113,536	41.7	272,256
West Virginia	46.4	1.54	140,491	24.9	71.0	2.45	177,749	31.4	80.4	3.33	247,173	43.7	565,413
Wisconsin	67.3	1.65	409,569	27.0	75.5	2.30	329,746	21.7	83.7	3.88	777,756	51.3	1,517,071
Totals and averages	60.4	1.72	3,871,621	21.5	68.2	2.82	4,233,401	23.6	71.5	4.48	9,877,104	54.9	17,982,126

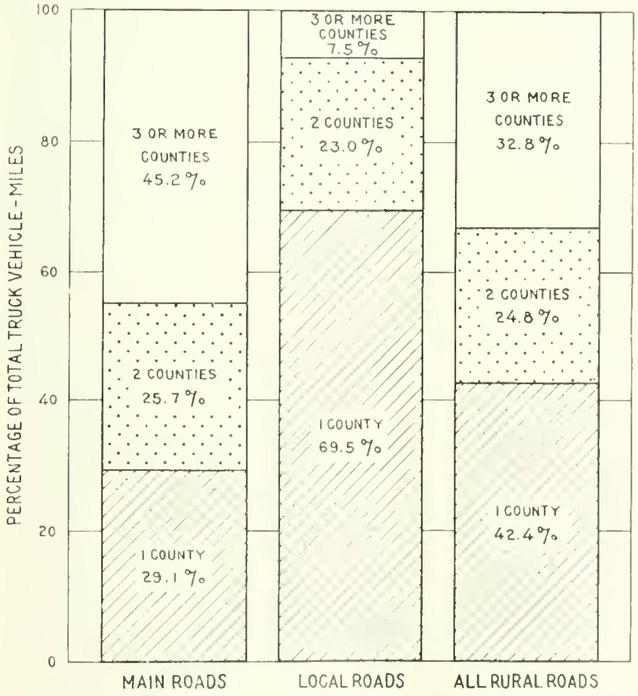

FIGURE 14.—PERCENTAGE DISTRIBUTION OF VEHICLE-MILES OF TRUCKS ON THE BASIS OF TRIP EXTENT IN TERMS OF NUMBER OF COUNTIES INVOLVED ON MAIN AND LOCAL RURAL ROADS IN 17 STATES IN THE YEAR 1940.

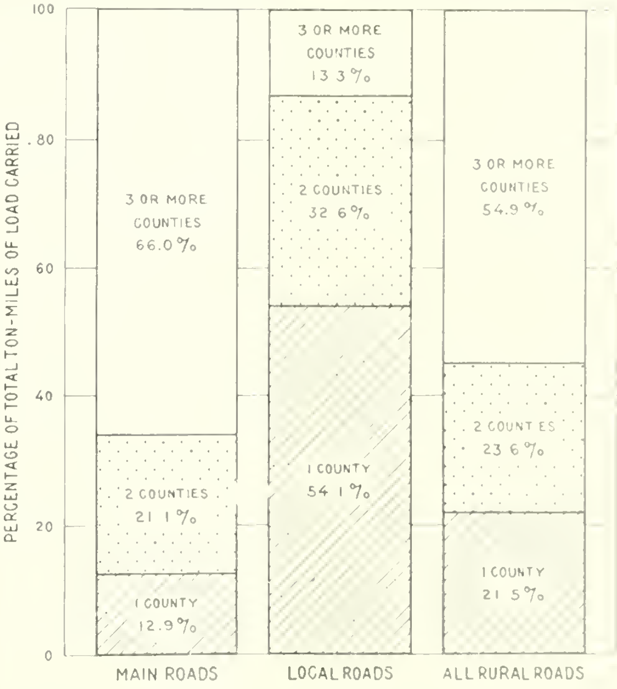

FIGURE 15.—PERCENTAGE DISTRIBUTION OF TON-MILES OF LOAD CARRIED BY TRUCKS ON THE BASIS OF TRIP EXTENT IN TERMS OF NUMBER OF COUNTIES, ON MAIN AND LOCAL ROADS IN 16 STATES IN THE YEAR 1940.

WARTIME TRENDS ESTIMATED

The estimates so far presented have related entirely to the year 1940. Since our entrance into the war, a number of things have happened that have had important effects upon the amount and characteristics of trucking on rural roads. Restrictions have been placed on the purchase of automobiles, tires, and gasoline; legal restrictions on sizes and weights of vehicles have been lifted or relaxed in some States; and regulations have been issued by the Office of Defense Transportation. Changes from a peacetime economy to a wartime economy have involved changes in the kind of materials hauled and in their origins and destinations. Hauling to cantonments and war plants during and after construction has increased to large proportions, while hauling in many other categories has decreased. All of these things have resulted in changes in traffic volumes, in the percentage relations between vehicle types and in the average weight of the load carried.

TABLE 22.—*Estimates by census regions, of 1936, 1940, and 1942 vehicle-mileage by all vehicles, truck vehicle-mileage, percentage loaded, average carried load, and ton-mileage of load hauled on MAIN RURAL ROADS*

Census region	Year	Travel by all vehicles	All trucks (including combinations)					Single-unit trucks					Truck combinations				
			Percentage of all vehicles	Travel by trucks and combinations	Percentage loaded	Carried load		Percentage of all trucks	Travel by single units	Percentage loaded	Carried load		Percentage of all trucks	Travel by combinations	Percentage loaded	Carried load	
						Average weight	Amount				Average weight	Amount				Average weight	Amount
		Thousands of vehicle-miles		*Thousands of vehicle-miles*		*Tons*	*Thousands of ton-miles*		*Thousands of ton-miles*		*Tons*	*Thousands of ton-miles*		*Thousands of ton-miles*		*Tons*	*Thousands of ton-miles*
New England	1936	5,252,578	12.7	665,157	68.0	2.91	1,314,315	88.8	590,803	66.6	2.21	869,145	11.2	74,354	79.4	7.54	445,170
	1940	6,310,944	14.7	926,387	71.7	3.30	2,194,376	86.9	805,330	70.6	2.49	1,412,240	13.1	121,057	79.4	8.14	782,136
	1942	4,246,210	17.6	746,171	60.7	4.44	2,010,824	75.6	563,960	55.2	2.70	839,631	24.4	182,211	77.8	8.26	1,171,193
Ratio 1942 to 1940		0.67	1.20	0.81	0.85	1.35	0.92	0.87	0.70	0.78	1.08	0.59	1.86	1.50	0.98	1.01	1.50
Middle Atlantic	1936	13,090,754	14.1	1,841,884	62.6	2.87	3,312,557	84.1	1,553,959	61.2	1.99	1,895,803	15.6	287,925	70.2	7.00	1,416,754
	1940	15,809,923	16.1	2,540,395	65.9	3.29	5,504,412	81.9	2,081,515	64.9	2.27	3,064,582	18.1	458,880	70.2	7.57	2,439,830
	1942	12,053,622	17.6	2,122,683	53.0	4.55	5,118,594	73.9	1,568,321	50.3	2.83	2,233,468	26.1	554,362	60.5	8.60	2,885,126
Ratio 1942 to 1940		0.76	1.09	0.84	0.80	1.38	0.93	0.90	0.75	0.78	1.25	0.73	1.44	1.21	0.86	1.14	1.18
South Atlantic	1936	12,475,419	19.6	2,449,370	61.2	3.23	4,836,079	81.1	1,986,540	59.2	2.18	2,558,069	18.9	462,830	69.8	7.05	2,278,010
	1940	16,536,492	21.5	3,563,118	64.3	3.67	8,416,522	77.7	2,766,916	62.8	2.40	4,172,832	22.3	796,202	69.8	7.64	4,243,690
	1942	13,121,142	24.7	3,241,733	59.2	4.45	8,547,979	71.4	2,316,192	56.7	2.33	3,058,287	28.6	925,541	65.5	9.06	5,489,692
Ratio 1942 to 1940		0.79	1.15	0.91	0.92	1.21	1.02	0.92	0.84	0.90	0.97	0.73	1.28	1.16	0.94	1.19	1.29
East North Central	1936	17,943,461	15.8	2,836,513	64.9	3.53	6,491,337	72.6	2,060,097	61.3	1.95	2,167,071	27.4	776,416	74.4	6.97	4,024,266
	1940	22,155,069	17.6	3,890,392	67.9	4.04	10,675,437	68.4	2,660,707	65.0	2.19	3,790,553	31.6	1,229,685	74.2	7.55	6,884,884
	1912	19,112,499	17.6	3,361,540	60.4	4.94	10,012,855	61.7	2,069,375	54.2	2.40	2,667,904	38.3	1,292,165	70.3	8.06	7,344,951
Ratio 1942 to 1940		0.86	1.00	0.86	0.89	1.22	0.94	0.90	0.78	0.83	1.09	0.70	1.21	1.05	0.95	1.07	1.07
East South Central	1936	5,699,938	20.5	1,168,149	60.2	2.46	1,730,235	91.5	1,068,311	59.4	2.14	1,357,462	8.5	99,838	68.7	5.44	372,773
	1940	7,241,058	22.4	1,621,392	63.5	2.68	2,753,110	90.0	1,458,710	62.9	2.28	2,095,985	10.0	162,682	68.6	5.89	657,125
	1942	6,607,466	22.1	1,458,587	48.4	3.41	2,404,214	85.3	1,244,842	45.4	2.41	1,359,976	14.7	213,745	65.7	7.44	1,044,238
Ratio 1942 to 1940		0.91	0.99	0.90	0.76	1.27	0.87	0.95	0.85	0.72	1.06	0.65	1.47	1.31	0.96	1.26	1.59
West North Central	1936	10,751,485	18.1	1,948,109	61.9	2.81	3,394,974	83.7	1,629,707	60.2	2.00	1,955,831	16.3	318,402	71.0	6.37	1,439,143
	1940	12,604,672	19.8	2,494,536	66.2	3.13	5,177,297	81.4	2,029,965	65.1	2.22	2,930,359	18.6	464,571	71.2	6.80	2,246,938
	1942	11,376,164	19.9	2,258,967	59.7	3.47	4,684,886	78.6	1,776,127	56.9	2.19	2,192,422	21.4	482,840	70.0	7.37	2,492,464
Ratio 1942 to 1940		0.90	1.01	0.91	0.90	1.11	0.90	0.97	0.87	0.87	0.99	0.75	1.15	1.04	0.98	1.08	1.11
West South Central	1936	9,857,964	19.1	1,879,481	57.4	2.28	2,456,025	82.0	1,541,603	55.4	1.67	1,426,580	18.0	337,878	66.9	4.56	1,029,445
	1940	12,043,568	21.5	2,587,225	60.4	2.58	3,964,048	79.2	2,048,629	58.7	1.82	2,188,694	20.8	538,596	66.9	4.93	1,775,354
	1942	11,312,122	20.6	2,334,281	49.6	3.45	3,994,521	74.7	1,744,700	44.9	1.95	1,523,190	25.3	589,581	63.6	6.59	2,471,331
Ratio 1942 to 1940		0.94	0.96	0.90	0.83	1.34	1.01	0.94	0.85	0.76	1.07	0.70	1.22	1.09	0.95	1.34	1.39
Mountain	1936	4,792,625	19.5	933,147	58.9	2.70	1,485,818	90.6	845,217	57.8	2.19	1,070,646	9.4	87,930	69.4	6.80	415,172
	1940	6,151,752	20.5	1,260,844	62.2	3.12	2,447,647	88.5	1,115,824	61.2	2.48	1,691,767	11.5	145,020	69.7	7.48	755,880
	1942	5,521,263	20.3	1,123,391	51.4	4.20	2,423,339	81.1	910,569	47.5	2.58	1,115,530	18.9	212,822	68.1	9.03	1,307,809
Ratio 1942 to 1940		0.90	0.99	0.89	0.83	1.35	0.99	0.92	0.82	0.78	1.04	0.66	1.64	1.47	0.98	1.21	1.73
Pacific	1936	8,823,900	14.4	1,272,187	64.9	4.29	3,544,040	79.6	1,012,878	61.5	2.07	1,287,756	20.4	259,309	78.0	11.16	2,256,284
	1940	10,776,142	15.5	1,674,017	68.1	4.87	5,552,872	76.5	1,280,211	65.1	2.23	1,859,789	23.5	393,806	77.9	12.05	3,693,083
	1942	9,821,024	16.3	1,601,787	67.9	6.22	6,762,354	73.7	1,181,114	64.0	2.33	1,756,838	26.3	420,673	79.0	15.05	5,005,516
Ratio 1942 to 1940		0.91	1.05	0.96	1.00	1.28	1.22	0.96	0.92	0.98	1.01	0.94	1.12	1.07	1.01	1.25	1.36
Total	1936	88,688,124	16.9	14,993,997	62.1	3.06	28,565,380	82.0	12,289,115	59.9	2.02	14,888,363	18.0	2,704,882	71.9	7.03	13,677,017
	1940	109,629,620	18.8	20,558,306	65.4	3.47	46,685,721	79.0	16,247,807	63.7	2.24	23,206,801	21.0	4,310,499	71.8	7.58	23,478,920
	1942	93,175,112	19.6	18,249,140	57.0	4.42	45,959,566	73.3	13,375,200	52.9	2.37	16,747,246	26.7	4,873,940	68.2	8.79	29,212,320
Ratio 1942 to 1940		0.85	1.04	0.89	0.87	1.27	0.98	0.93	0.82	0.83	1.06	0.72	1.27	1.13	0.95	1.16	1.24

Wartime changes in traffic volume, composition, and weights on main rural roads are shown in table 22 which gives estimates for the years 1936, 1940, and 1942 for each of the census regions of the United States shown in figure 16. The method of preparing the 1940 estimates has been previously described and, in cases where the original survey was made in a year other than 1936, the 1936 estimates were prepared in a similar manner, projecting backwards to 1936 the trends indicated by the best available data for each item. Since the original survey was made in either 1936 or 1937 in all but a few States, only slight adjustments, if any, were generally required to prepare 1936 estimates from the survey year data.

The 1942 estimates are based in part on data from the regular continuing survey operations and in part on the results of a special short survey made in July and August 1942, by 46 State highway departments in cooperation with the Public Roads Administration. The vehicle-mileage by all vehicles was computed on the basis of trends established by 466 automatic traffic recorders operated continuously throughout 1940 and 1942. The truck-mileage was determined on the basis of the trends found to exist in the mid-summer survey, but classification counts in 16 States made at the same 122 stations in all seasons of 1940 and 1942 checked the trend in the United States total almost exactly. These seasonal classification counts were well distributed geographically, but did not give sufficient coverage in any one region to justify a check by regions. The 1942 estimates for the percentage distribution between single-unit trucks and combinations, percentage loaded, and average weight, are based entirely on trends established by comparing data on over 50,000 trucks taken at 486 stations in the 1942 survey with corresponding data taken at the identical locations in a comparable period in the same season of a previous year.

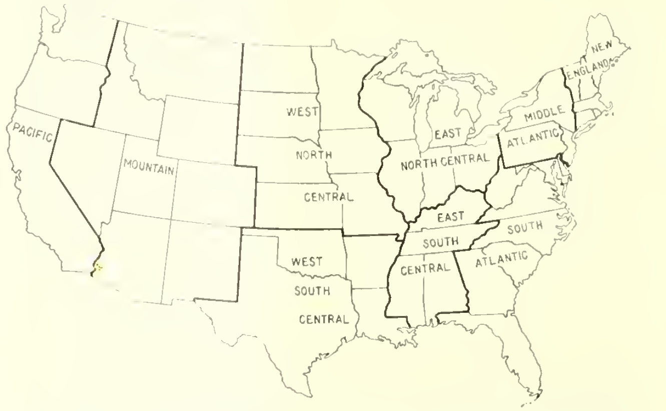

FIGURE 16.—CENSUS REGIONS OF THE UNITED STATES.

Table 22 was limited to data on main roads because of the lack of figures on truck loading on local roads in 1942. However, gasoline consumption figures, automatic traffic recorder data, and classification counts were sufficient to permit the preparation of approximate esti-

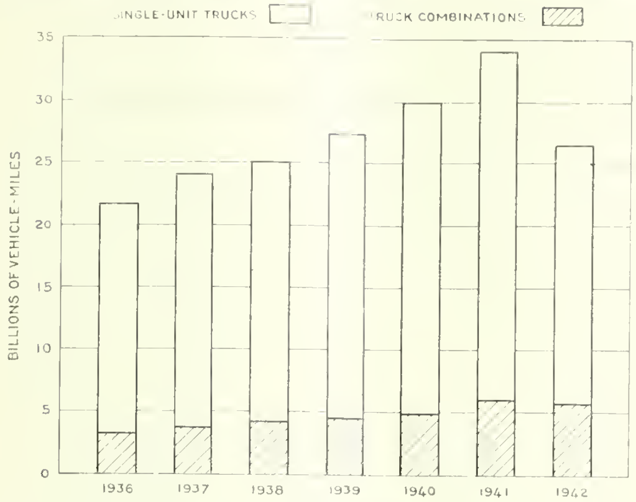

Figure 17.—Estimated Vehicle-Miles of Single-Unit Trucks and of Combinations on All Rural Roads in the United States in Each Year From 1936 to 1942, Inclusive.

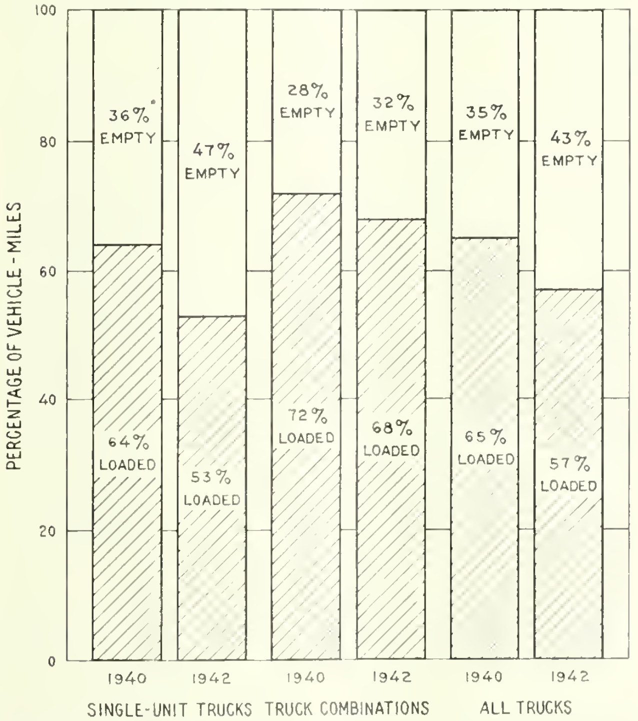

Figure 18.—Comparison of Percentage of Vehicle-Miles Loaded in 1942 With That in 1940, For Single-Unit Trucks and Combinations on Main Rural Roads Throughout the United States.

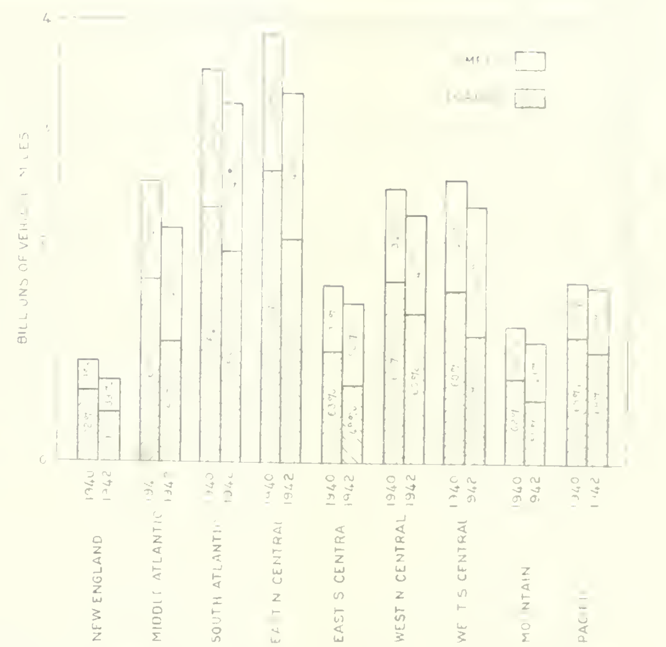

Figure 19.—Comparison of Estimated Vehicle-Miles of Empty Trucks and of Loaded Trucks on Main Rural Roads in 1942 With Those in 1940, in Each Census Region.

mates of the mileage traveled by single-unit trucks and by combinations on all rural roads in each year from 1936 to 1942, inclusive. These estimates are presented in figure 17, which is the only figure showing wartime trends which apply to all rural roads. It shows that the mileage of truck travel of all kinds increased steadily from 1936 to 1941, but dropped sharply in 1942, while the mileage of combinations increased steadily from 1936 to 1942 with a slight peak above the trend line in 1941. Throughout this period, the tendency was toward an increased use of combinations, particularly tractor-truck and semitrailer combinations, such as are shown in figure 1, and this tendency increased considerably under war conditions. Table 22 shows that, on main roads, combinations were 18.0 percent of all trucks in 1936, 21.0 percent in 1940, and 26.7 percent in 1942.

TRAFFIC BY EMPTY TRUCKS INCREASED WHILE THAT BY LOADED TRUCKS DECREASED

One of the surprising facts discovered in the 1942 survey was that the percentages of both single-unit trucks and combinations running empty were higher in 1942 than in previous years, in spite of the efforts of the Office of Defense Transportation to bring about fuller utilization of truck capacity. Figure 18 shows that the percentage of empty single-unit trucks increased to a considerable extent, while the percentage of empty combinations increased to a lesser extent. The number of loaded single-unit trucks using the main rural roads in 1942 was not much greater than the number of empty trucks of this type.

One reason for this relative increase in empty trucks is that trucks hauling materials to construction jobs, or supplies to cantonments, generally cannot obtain return loads. Hauling in these categories increased, while the hauling of goods between cities, a type of hauling in which two-way loads are relatively easy to obtain, decreased. This does not fully explain the very high percentage of empty single-unit trucks, however, which was 50 percent or more in a number of States. It seems probable that there was a considerable use of light trucks to transport people, caused by restrictions on the purchase of new automobiles, tires, and gasoline. Apparently, many farmers and other persons owning both trucks and passenger cars put away their passenger cars and used their trucks for all purposes.

Figure 19 shows the mileage traveled by loaded and empty trucks of all kinds in each census region in 1940 and in 1942. Figures in the bars indicate the percentage of vehicles loaded and the percentage empty, in each case. In all regions, total truck traffic and movement of loaded trucks were less in 1942 than in 1940. In all except the Pacific region, mileage of empty trucks increased, while that of loaded trucks decreased. In the East South Central States the mileage of empty

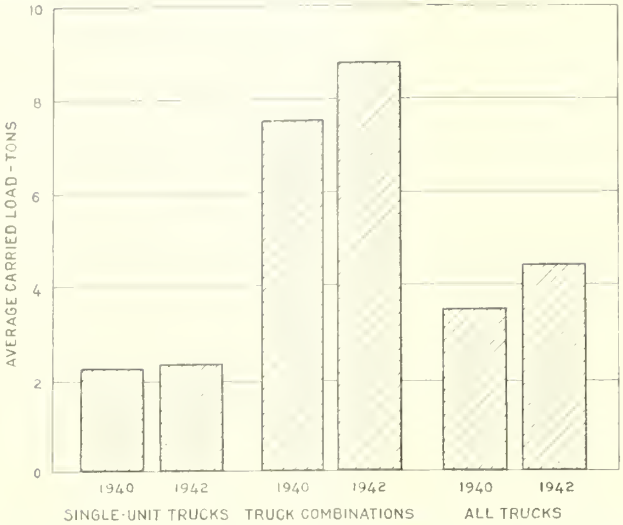

Figure 20.—Comparison of Estimated Average Carried Load of Loaded Single-Unit Trucks and Combinations in 1942 With That in 1940, on Main Rural Roads Throughout the United States.

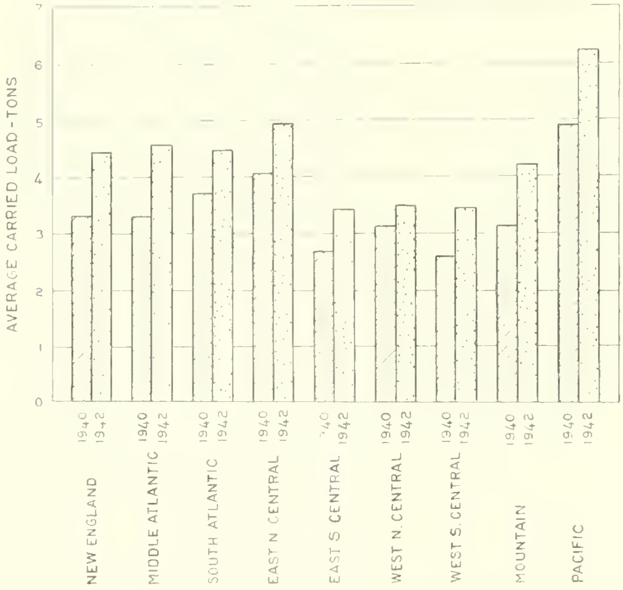

Figure 21.—Comparison of Estimated Average Carried Load of Loaded Trucks in 1942 With That in 1940, on Main Rural Roads in Each Census Region.

trucks actually exceeded that of loaded trucks in 1942, and in the West South Central States the truck traffic was divided equally between empty and loaded vehicles.

SMALLER VEHICLE-MILEAGE OFFSET BY HEAVIER LOADS

Figure 20 shows that the average load carried by loaded vehicles was greater in 1942 than in 1940 for both single-unit trucks and for combinations. Because of the increased proportion of combinations, the average load of loaded vehicles of both types combined increased by a greater percentage than that of either type. This increase in average load took place in all regions, as shown by figure 21.

For main roads in the United States as a whole, the decrease in mileage of loaded vehicles was nearly offset by the increase in carried load so that the ton-mileage was almost as great in 1942 as in 1940. Figure 22 is drawn with the vehicle-mileages of loaded vehicles as abscissas and the average carried loads as ordinates so that ton-mileages are represented by rectangular areas. The rectangle for 1942 is shorter than that for 1940, but is higher by an amount that makes it almost equal in area.

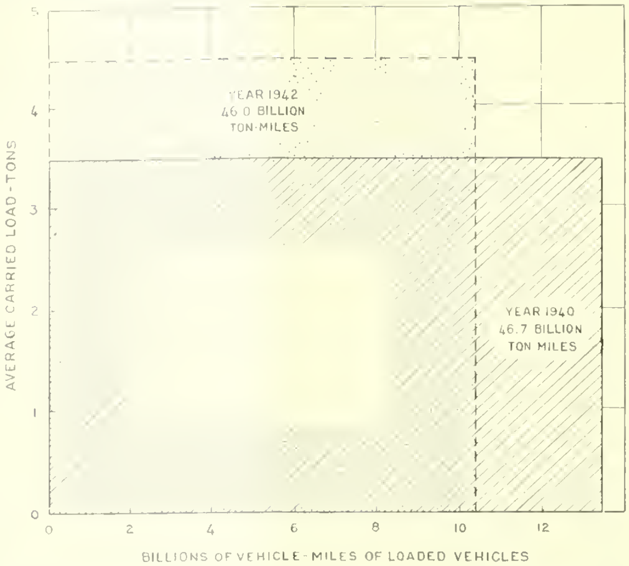

Figure 22.—Comparison of Estimated Ton-Miles in 1942 With Those in 1940, on Main Rural Roads in the United States.

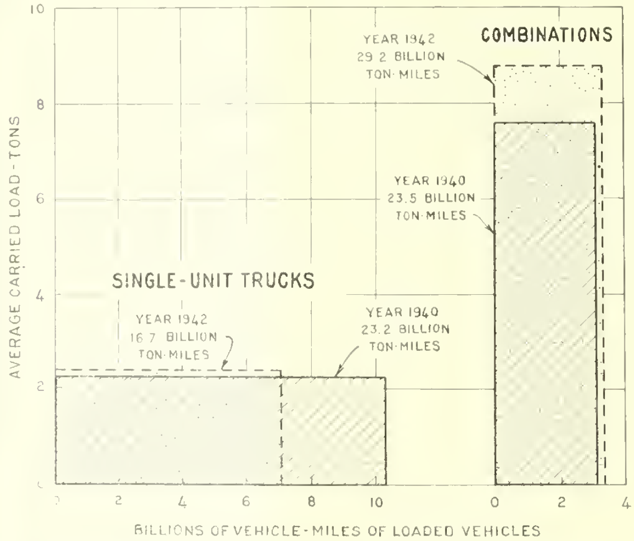

Figure 23.—Comparison of Estimated Ton-Miles Carried by Single-Unit Trucks and by Combinations in 1942 With Those in 1940, on Main Rural Roads in the United States.

A study of these relations by vehicle type emphasizes the increasing importance of combinations in trucking on main rural roads. Figure 23 is constructed in the same manner as figure 22, but separate ton-mile rectangles are shown for single-unit trucks and for combinations. From 1940 to 1942, the mileage of loaded single-unit trucks decreased considerably and the average carried load increased only slightly with a resultant large loss in ton-mileage by vehicles of this type. On the other hand, both the mileage of loaded vehicles and the average of loads carried by combinations increased and the ton-mileage increased correspondingly. In 1940, the ton-mileage by combinations was about the same as that by single-unit trucks, while in 1942 the

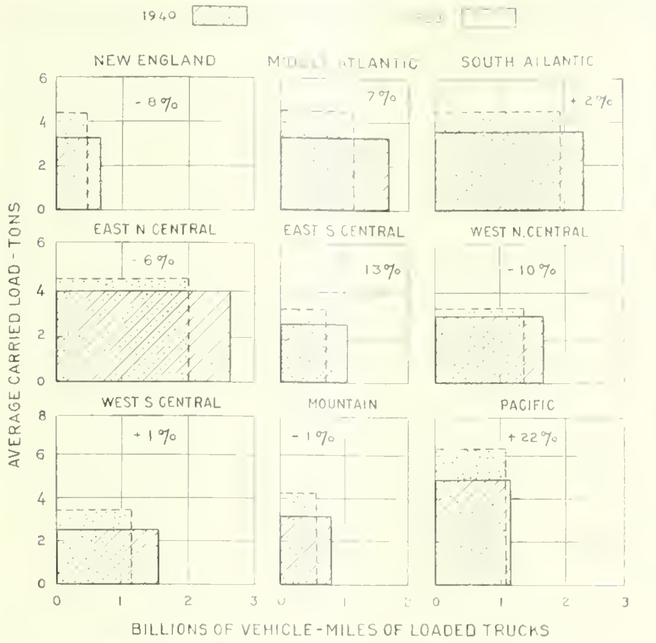

Figure 24.—Comparison of Estimated Ton-Miles in 1942 With Those in 1940, on Main Rural Roads in Each Census Region.

combinations carried about 74 percent more than the single-unit trucks.

Figure 24 shows, by means of rectangles, the ton-mileage by all trucks, including combinations, on main rural roads in each census region, in 1940 and 1942. The figures above the rectangles indicate the percentage increase or decrease in ton-mileage from 1940 to 1942. Only in the Pacific region was there a substantial increase. In two of the other regions there was a negligible increase, and in the other six regions there was a decrease.

There is considerable evidence to indicate that the tonnages hauled by large truckers were much greater in 1942 than in 1940. Reports to the Interstate Commerce Commission and to the American Trucking Associations, Inc., tell of increases of this kind. This is not necessarily inconsistent with the conclusion that the ton-mileage by all trucks did not increase from 1940 to 1942. As has already been brought out, the large truckers making periodic reports constitute a very small percentage of the total. It is entirely possible that much of their increased tonnage was diverted from truck owners who preferred to ship by common carrier rather than use their own trucks because of the difficulty in obtaining new vehicles and parts. The fact that both vehicle-mileage and ton-mileage increased for combinations and decreased for single-unit trucks tends to support such a conclusion.

Another fact to be borne in mind is that increased tonnages do not necessarily mean increased ton-mileages, as the average length of haul might be reduced. Shipments normally sent all of the way from origin to destination by truck might, under war conditions, go by truck only to or from the nearest railroad station. Also, some trucks which formerly made long trips between cities may have been diverted to shorter trips between rail or water terminals or sources of supply and construction jobs, cantonments, and war plants.

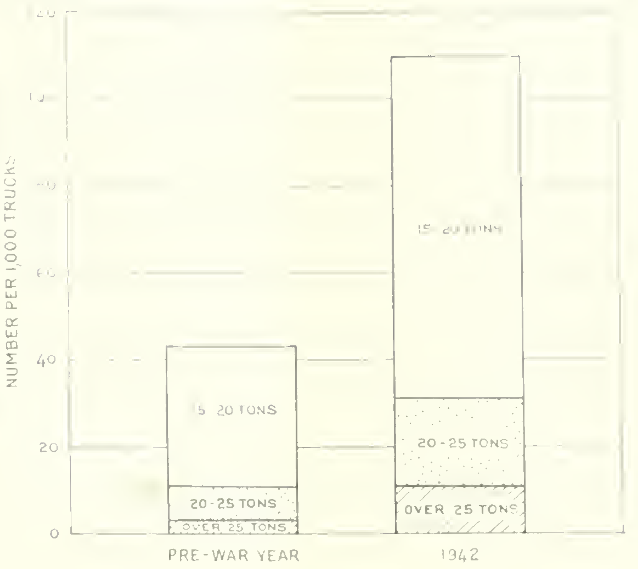

Figure 25.—Number of Heavy Gross Loads Per 1,000 Loaded and Empty Trucks in the Summer of 1942 and in a Corresponding Period of a Pre-War Year Between 1936 and 1940, on Main Rural Roads Throughout the United States.

HEAVY GROSS LOADS AND AXLE LOADS INCREASED IN FREQUENCY

From the point of view of the highway engineer, the frequencies of heavy gross loads, axle loads, and load concentrations are more important than averages, because these are the frequencies of severe stressing of road surfaces and bridges and are the basis for design. It has already been shown that averages of loads carried increased materially between 1940 and 1942, and it is reasonable to assume that the frequency of heavy gross weights increased correspondingly during this period. Available trend data were not sufficient to justify estimates of the frequencies of heavy gross weights and axle loads for the year 1940, so the 1942 frequencies were compared with pre-war averages, determined by averaging data taken in the original survey regardless of the survey year. This combining of data taken in different years in different States, of course, gives only an indication of average conditions existing in the 1936 to 1940 period, with the earlier years of the period given more weight than the latter years since the original survey was in 1936 or 1937 in most States. Though the pre-war average is therefore somewhat vague as to meaning, it is nevertheless a useful figure as a basis for determining the wartime changes in the frequencies of heavy axle loads in different parts of the country.

Figure 25 shows that heavy gross loads were much more frequent in 1942 than in the pre-war period. For each 1,000 trucks found on main roads, an average of 110 weighed over 15 tons in 1942 compared to an average of 43 weighing over 15 tons in the 1936-1940 period. Trucks weighing over 25 tons averaged 11 per thousand in 1942 compared to only 3 per thousand in the pre-war period.

This increase in the frequency of heavy gross loads took place in all regions, as is shown by figure 26. In 1942, gross loads over 15 tons were most frequent in the East North Central region, but nearly all of these were between 15 and 20 tons or between 20 and 25 tons

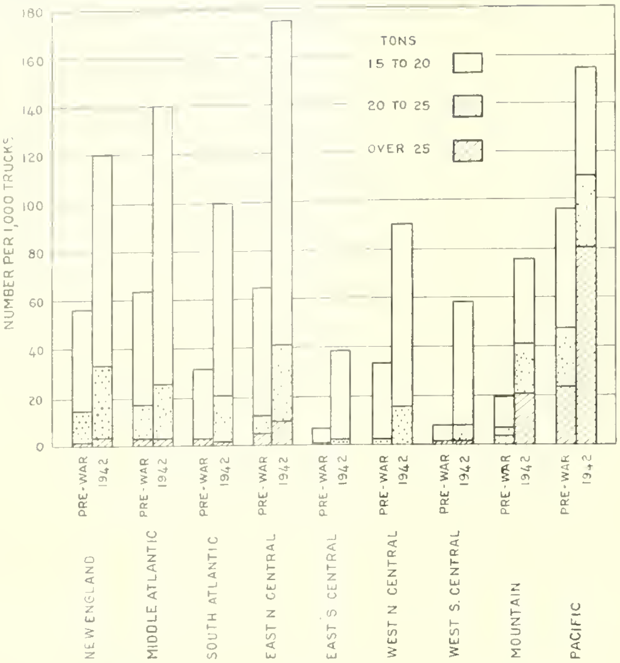

Figure 26.—Number of Heavy Gross Loads Per 1,000 Loaded and Empty Trucks in the Summer of 1942 and in a Corresponding Period of a Pre-War Year Between 1936 and 1940, on Main Rural Roads in Each Census Region.

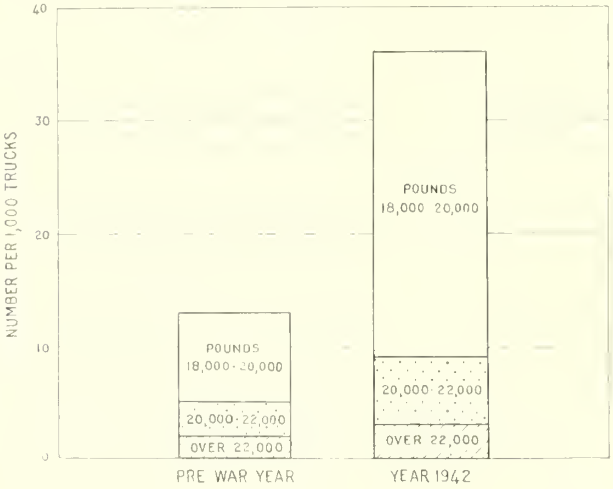

Figure 27.—Number of Heavy Axle Loads Per 1,000 Loaded and Empty Trucks in the Summer of 1942 and in a Corresponding Period of a Pre-War Year Between 1936 and 1940, on Main Rural Roads Throughout the United States.

The frequency of gross loads over 25 tons was by far the greatest in the Pacific region, due to the use in this region of large combinations, frequently consisting of a tractor-truck, a semitrailer, and a trailer (fig. 1D). Most of the combinations in the East North Central States were of the type shown in figure 1A.

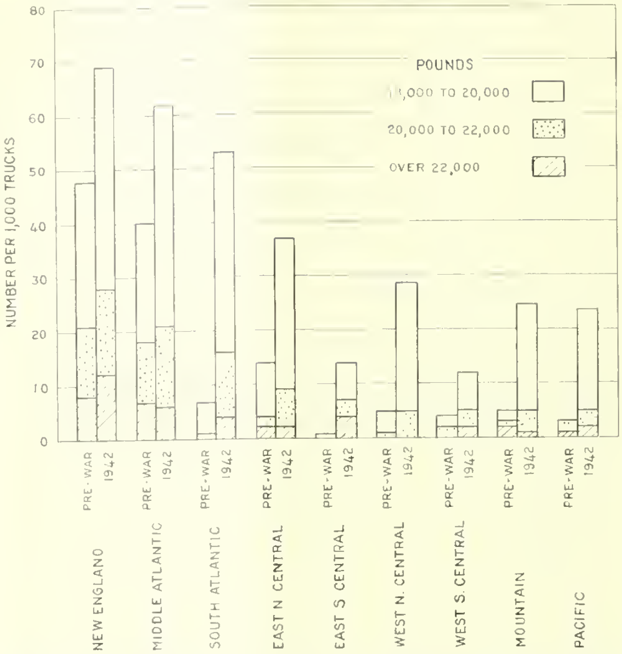

Figure 28.—Number of Heavy Axle Loads Per 1,000 Loaded and Empty Trucks in the Summer of 1942, and in a Corresponding Period of a Pre-War Year Between 1936 and 1940, on Main Rural Roads in Each Census Region.

The increase in frequency of heavy gross weights does not necessarily mean an increase in the frequency of heavy axle loads, since the larger vehicles generally have more axles than the smaller ones. However, heavy axle loads were much more frequent in 1942 than in the pre-war period, as is shown by figure 27. For each 1,000 trucks passing over the main roads there were, on the average, 36 axles weighing over 18,000 pounds in 1942 compared to 13 in the pre-war period. There were similar increases in the frequency of axle loads over 20,000 pounds and over 22,000 pounds.

All regions had a greater frequency of heavy axle loads in 1942 than in the pre-war period. Figure 28 shows that the increase was very great in some regions such as, for example, in the South Atlantic States, where axle loads over 18,000 pounds increased from 7 per thousand trucks in the pre-war period to 53 per thousand trucks in 1942. It is rather surprising to find from this figure that the New England States had the greatest frequency of heavy axle loads in 1942, since figure 26 showed that the frequency of heavy gross loads in the New England region was little, if any, above the national average. On the other hand, the Pacific region, which had such a high frequency of gross loads over 25 tons, was well below the national average in frequency of heavy axle loads. The explanation lies in the fact that the large combinations, which are confined mainly to the west, have their loads well distributed on a number of axles. For example, the three-axle tractor-truck with triple-axle semitrailer, shown in figure 1C, weighed 64,430 pounds, but no axle load was more than 12,020 pounds. The tractor-truck, semitrailer, and trailer combination shown in figure 1D weighed 72,255 pounds, but no axle load was more than 14,010 pounds. In contrast to this, the single-unit truck shown in figure 3A has most of its weight on one axle, and the one shown in figure 3B, while better proportioned, still had about two-thirds of its weight on the rear axle. The cab-over-engine truck in figure 3C and the truck with dual rear axles in figure 3D have better weight distribution. The laws in different States have much to do with the extent of usage of vehicles of different types.

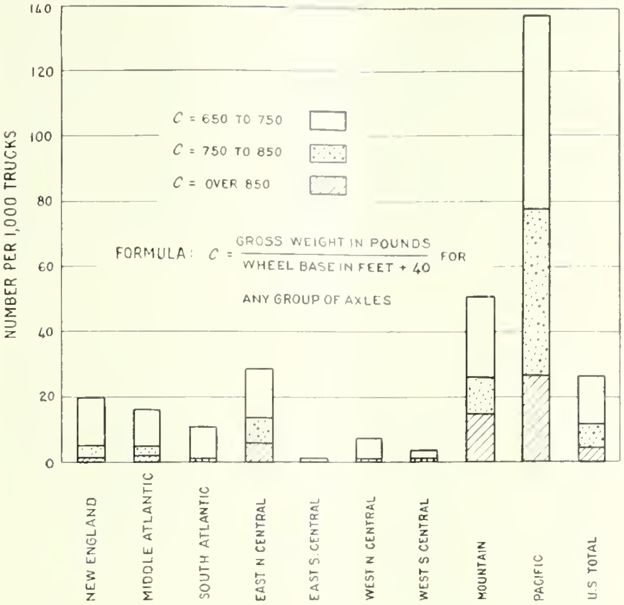

FIGURE 29.—NUMBER OF TRUCKS PER 1,000 LOADED AND EMPTY TRUCKS WITH HIGH VALUES OF C IN THE GROSS-WEIGHT FORMULA, IN THE SUMMER OF 1942, ON MAIN RURAL ROADS IN EACH CENSUS REGION.

HEAVIEST LOAD CONCENTRATIONS GENERALLY PRODUCED BY INTERIOR GROUPS OF AXLES

While the large combinations prevalent on the Pacific coast do not impose heavy axle loads, they do have heavy load concentrations on two or more axles that produce high bridge stresses.

The gross load formula $C=W\div(L+40)$ has been designed to give an indication of the degree of load concentration, which might be more important from the point of view of bridge stresses than either the gross load or the axle load. In this formula L is the distance in feet between the first and last axle of the vehicle, or of any interior group of axles; W is the total weight of the vehicle or the weight transmitted by the interior group of axles; and C is a measure of the load concentration. Numerous States have laws limiting the maximum permissible value of C for any load to a specific value. The value is different in different States, and sometimes depends upon axle spacing but ordinarily is 750, 700, or 650.

Since complete information on axle spacing was not obtained in the original survey, it was not possible to establish trends in values of C, but figure 29 shows the frequencies of various values as found in the summer of 1942. It will be noted that values in excess of 750 were very frequent in the Mountain and Pacific regions and occurred to a lesser extent in other regions.

It is not the over-all wheel base, but some interior group of axles that generally has the highest value of C for a truck with more than two axles. This fact is especially significant, since the laws of some States provide that only the over-all wheel base shall be considered in calculating C.

Table 23, based on 1942 observations, shows that, of the 542 vehicles with a value of C greater than 750, only 15 were so designed and loaded that the highest value of C was derived from consideration of over-all wheel base. In 468 cases, or 86 percent of the total, consideration of all axles other than the front axle yielded the highest value.

TABLE 23.—*Number of trucks observed in the summer of 1942, having values of C in the gross weight formula greater than 750, classified in accordance with the axle group having the highest value of C*[1]

Type of vehicle	Axle group with highest value of C			
	Over-all wheel base	Second to last axle	Other group	Total
Single-unit (2-axle)	1	------	------	1
Single-unit (3-axle)	1	31		32
Combinations	13	437	59	509
Total	15	468	59	542

[1] The gross weight formula is $C=\frac{W}{L+40}$ in which L is the distance in feet between the first and the last axle of the vehicle or of any interior group of axles, and W is the total weight of the vehicle or of the interior group of axles.

APPENDIX.—SOURCES OF DATA AND METHODS OF COMPUTATION

For the benefit of those who may wish to carry the estimates further, and prepare comparable estimates of some of the items for other years, or to determine the extent to which the various items are based on ample data, the sources of the data and the methods of arriving at the figures will be described in some detail.

HIGHWAY PLANNING SURVEYS DESCRIBED

The estimates are founded primarily on the numerous tables of various series prepared as a part of the highway planning surveys conducted by all States in cooperation with the Public Roads Administration within the past few years. The original field surveys were made in most States in 1936 or 1937, but surveys were not made until 1938, 1939, or 1940 in some States, and in a few States they have not yet been completed. These surveys were very extensive and had many ramifications that are not pertinent to the estimates. Only the collection of information having an important bearing on the amount and characteristics of trucking on rural roads, will be briefly described.

In the rural road inventory, all roads outside of municipalities, or closely built-up urban communities, which were open to unrestricted public use, were driven over and measured with automobile odometers. The mileage figures thus determined were used in the calculation of vehicle-miles.

Traffic was counted at about 350,000 points on rural roads of all classes. At some of these points the count was for 8 hours only; at others there were from four 8-hour counts to twelve 24-hour counts, seasonally spaced. At about 8,000 important road intersections there were from eighteen to twenty-one 8-hour counts distributed throughout the seasons of the year, the days of the week, and the hours of the day in such manner as to permit the determination of average daily traffic for the approximately 28,000 road sections radiating from the intersections, and the development of traffic patterns to aid in the expansion of the shorter counts to daily averages. Automatic traffic recorders of fixed and portable types aided further in the determination of traffic patterns. From this combined information, it was possible to estimate average daily traffic throughout the year on every mile of public road, and to calculate vehicle-miles by multiplying by the mileage obtained in the inventory. In all of the traffic counts except those made by automatic recorders a separate classification was made of single-unit trucks, tractor-trucks with semitrailers, and truck and trailer combinations so that, in general, it was possible to calculate the vehicle-miles of trucks and combinations on main and local roads on the basis of ample data.

In the so-called "road-use" survey, which was a part of the highway planning surveys, representative motor-vehicle owners were questioned concerning the amount of driving done on roads of different classes during the preceding year. Approximately 185,000 truck owners in 44 States were interviewed. From their answers it was possible to calculate the truck-miles driven on rural roads. These calculations served to check the calculations based on the traffic survey, and to supply vehicle-mile figures for a few States that had not completed the traffic tables.

At about 3,000 of the traffic-count stations, trucks were weighed on portable scales known as "loadometers," and their drivers were questioned. The loadometer schedules called for operation in all seasons, on all days of the week, and at all hours of the day and night. Included in the information obtained was the following: The vehicle type and capacity, whether the vehicle was loaded or empty; the total weight of the vehicle; the weight of the carried load or the empty weight of loaded vehicles, if obtainable; and the origin and destination of the trip. This information was obtained for over 2,500,000 trucks, mostly on main roads, though, in some cases, vehicles traveling on local roads intersecting a main road at a loadometer station were sampled. Except for those States that had not completed loadometer tables, the weight data were fully adequate for computing ton-miles hauled on main roads in the survey year, in the aggregate, and in each of a number of origin and destination categories.

In a number of States the origin and destination data obtained on main roads at loadometer stations were supplemented by data obtained at traffic-count stations on local roads, either by interviewing drivers, or by handing them questionnaires to be filled out and returned later. Eighteen States have prepared origin and destination tables for local roads, and the total local-road truck sample for these States was about 850,000.

SURVEYS TO ESTABLISH TRENDS

Since the original field surveys were completed, continuing operations have supplied data on trends for some phases of the survey. Changes in road mileages are reported annually. Through the continuous operation of over 650 automatic traffic recorders at permanent locations, the operation on schedule of about 2,000 portable recorders, and manual counts in which vehicles are classified by types, traffic trends are fairly well established, especially on main roads.

Unfortunately, few States undertook repeat weighing operations in the 1936–40 period which could be used in making the estimates for 1940. In order to determine wartime trends, however, 486 loadometer stations were operated in the summer of 1942 on main roads in 46 States. About 120,000 trucks were counted and classified, and about 53,000 were weighed in this survey.

In most States, 10 stations were selected so that the combined data would be representative of traffic on main rural roads and were operated for 8 hours each, either from 6 a. m. to 2 p. m. or from 2 p. m. to 10 p. m. on a weekday. To insure comparability with earlier data, and to permit determination of trends, stations operated in the original highway planning survey were selected, and the hours of operation at each station were the same as those for a weekday operation in the original survey within a few weeks of the same time of year. Stations were not operated on Saturday, Sunday, and at night (10 p. m. to 6 a. m.).

BASIC VEHICLE-MILEAGE AND TON-MILEAGE TABLES

Sources of data.—All available data pertinent to each portion of the estimate were examined, and an effort was made to explain discrepancies before the final figures were decided upon. The principal series of tables and other data examined were as follows:

1. Road-inventory tables (RI series).
2. Annual highway-mileage tables (SM and LM series).
3. Traffic-inventory tables (T series).
4. Origin-destination tables (OD series).
5. Loadometer tables (L series).
6. Pit-scale tables (P series).
7. Road-use tables (RU series).
8. Analysis of motor-fuel usage (Public Roads Administration table G–21 for different years).
9. Numerous special tables submitted by individual States such as: Michigan blanket-count tables for local roads (B series); Iowa traffic survey table T–101 for different years; Iowa 1940 loadometer series; annual traffic tabulations for Virginia, New Mexico, and other States; et cetera.
10. Highway planning survey reports published by numerous States.
11. Special tabulations entitled "Rural Road Usage by Traffic Types," based in part on the traffic survey and in part on the road-use survey.
12. Special tabulations entitled "Percentage distribution of rural vehicle-miles classified by vehicle type and by local and foreign registration."
13. Automatic traffic-recorder-station classification-count data, submitted to the Public Roads Administration periodically.
14. Letters from a number of States written in reply to specific inquiries.
15. Replies to telegrams of May 19, 1942, making inquiry concerning wartime changes in truck-traffic volumes and weights. While most of these replies treat with a period subsequent to 1940, some of them give data which aid in establishing trends prior to 1940.
16. Tables made as a part of the special 1942 survey to determine wartime trends (W series).

For many States, information on total vehicle-miles and on the percentage of trucks and combinations was available both from the traffic survey and the road-use survey. In such cases the traffic-survey figures were used because they include traffic by foreign vehicles, which is not included in the road-use figures. The figures from the two sources were compared, however, and were found to be reasonably consistent in all cases. Where only road-use figures were available, they were used after adjustment for travel by foreign vehicles.

Highway systems.—Since trucking characteristics on local roads differ materially from those on main roads, it was thought advisable to prepare separate estimates for the two classes of roads. The main roads in each State were well covered by loadometer stations, but relatively few loadometer data were obtained for local roads. Likewise, some of the trend information is applicable to the main roads only. It follows that the estimates for the main roads should be fairly accurate, while those for the local roads are only approximate. Since the local roads have less than 30 percent of the total vehicle-mileage and only about 20 percent of the total ton-mileage, an error of considerable proportions in the estimates for local roads would result in only a small percentage of error in the estimates for all rural roads.

In selecting the road mileage to be classed as "main" and that to be classed as "local," an effort was made to confine the main road mileage to that well covered by the loadometer survey. In most States this meant classifying the roads of the State system, or the primary State system, as main roads and all others as local roads. In Louisiana the classification was made on the basis of traffic volume rather than administrative system because a special study provided the basis for such a classification and indicated it to be preferable to the administrative classification. It will be noted that the term "local roads" as used in this analysis includes important county roads, secondary State roads, and other groups of roads not ordinarily called local.

Table 1 shows the characteristics of the two road systems as selected in each State. For the country as a whole, the main roads constituted 11.6 percent of the total rural mileage, carried 71.6 percent of the total rural vehicle-mileage, and had an average traffic density of 883 vehicles per day. The local roads constituted 88.4 percent of the mileage, carried 28.4 percent of the vehicle-mileage, and had an average traffic density of 46 vehicles per day. Table 1 serves to define the two systems and was included principally for this purpose.

Vehicle-mileage of all vehicles.—The figures for vehicle-miles of all vehicles in the survey year came from tables based on the traffic survey, with the following exceptions:

1. Figures for travel on the secondary systems in Arkansas and Illinois are from the road-use survey.
2. Figures for travel on both systems in Montana, Washington, and Wyoming, are in part from the traffic survey and in part from the road-use survey, as combined by the State.
3. Travel on the secondary system in Mississippi was estimated as explained in the next paragraph.
4. Delaware, Georgia, Maine, New Jersey, and New York have not submitted traffic or road-use tables showing vehicle-miles. Each has submitted either measured or estimated road-mileage figures. Average daily traffic figures were estimated for each road system in each of these States on the basis of corresponding figures for other States, similarly situated. Vehicle-miles were computed by multiplying the mileage by the average daily traffic so estimated. Data are not shown separately in the estimates for these States, with the exception of Georgia, but were grouped in an "Other States" classification. Data for Georgia were shown separately, because the availability of weight data made it possible to extend further the estimates for that State.

The vehicle-mile figures for 1940 were estimated from those for the survey year on the assumption that traffic on both systems increased in the same ratio as did the consumption of gasoline on highways in each State. The gasoline-consumption figures were obtained from Public Roads Administration table G-21. Where the survey year was made up of parts of two calendar years, the number of months in each calendar year was multiplied by one-twelfth of the gasoline consumption for that year and the two products were added to obtain the approximate gasoline consumption for the survey year.

Trucks and combinations as a percentage of all vehicles.—The vehicle-mileage of trucks and combinations expressed as a percentage of the vehicle-mileage of all vehicles on main roads in the survey year was submitted by 40 States in tables based on the traffic survey, or on a combination of the traffic and road-use surveys. Only a figure based on the road-use survey was available for Connecticut and this was multiplied by 0.91 which was the average ratio of the traffic-survey figure to the road-use figure for main roads in the 34 States for which both sets of figures were available. Since almost all trucks and combinations carry a license plate for the State in which they are operating and were therefore classed as local, the road-use survey, which includes only traffic by local vehicles, naturally shows a higher percentage for trucks and combinations than the traffic survey which includes foreign traffic, nearly all of which consists of passenger cars.

No information on the percentage for trucks and combinations on the primary system was submitted by Delaware, Georgia, Maine, Massachusetts, New Jersey, New York, or Rhode Island, and percentages were estimated for these seven States on the basis of corresponding percentages for other States similarly situated.

For local roads, 38 States submitted percentages for trucks and combinations based on the traffic survey, or a combination of the traffic and road-use surveys. Road-use figures for Connecticut and Vermont were multiplied by 0.97 to correct for foreign traffic, the correction factor being determined in the same manner as that for main roads. The factor is nearer unity for local roads than for main roads, because there are relatively fewer foreign vehicles on local roads. No percentages for trucks and combinations on local roads were submitted by Delaware, Georgia, Maine, Massachusetts, Mississippi, New Jersey, New York, and Rhode Island and percentages were estimated for these eight States on the basis of corresponding percentages for other States similarly situated.

Data permitting a computation of the trend in percentage for trucks and combinations on main roads between 1936 and 1940 were submitted by Arizona, Arkansas, California, Iowa, Missouri, Oklahoma, Oregon, South Dakota, and Wyoming. According to these data, the percentage declined very slightly between 1936 and 1940 in Arizona and Wyoming, and increased substantially in the other seven States. For the nine States, the percentage increased an average of 0.52 per year. Long-range trend data in Virginia and Nevada going back to 1926, show a steady increase averaging about 0.30 per year in Virginia and 0.55 in Nevada. Automatic traffic-recorder classification counts on primary roads throughout the United States show an increase of 0.7 from 1939 to 1940.

The only trend data available for local roads are from Arkansas, Missouri, Oklahoma, and Oregon. These data show an average increase in the percentage for trucks and combinations of 0.38 per year during the 1936–1940 period.

Considering all of the above data, the percentage for trucks and combinations on both systems was increased by 0.50 for each year intervening between the latest year for which a figure was available and 1940 to obtain the 1940 estimates. For example, if the percentage was 16.0 in 1936, it was considered to be 18.0 in 1940. The adjustment was made by adding an increment to the percentage, instead of multiplying the percentage by a factor as in the other adjustments, because this procedure appeared to be more consistent with the available data.

Combinations as a percentage of all trucks.—Data from the traffic surveys on the percentages for single-unit trucks and combinations, respectively, on main roads were submitted by 31 States. These figures are considered to be more accurate than those of the loadometer survey because the samples are much larger and are better distributed. Only the distribution from the loadometer sample was available for Georgia, Massachusetts, Minnesota, New Mexico, North Carolina, North Dakota, Ohio, Rhode Island, Texas, Utah, Vermont, and Wisconsin.

Twenty-eight States submitted data from traffic density surveys on local roads showing the percentage distribution between single-unit trucks and combinations. For these States combined, the percentage for combinations on local roads was about 0.3 the percentage on main roads. The percentages for main roads were therefore multiplied by 0.3 to estimate percentages for local roads in the 15 States for which main-road percentages only were known. These States were as follows: Georgia, Massachusetts, Minnesota, Mississippi, New Mexico, North Carolina, North Dakota, Ohio, Rhode Island, South Carolina, Texas, Utah, Vermont, Virginia, and Wisconsin.

Data showing the trend in the percentage for combinations between 1936 and 1940 were submitted by Arizona, Iowa, Missouri, and Virginia. All of these data indicate that the proportion of combinations in total truck traffic increased over this period. The average rate of increase in the four States was 4 percent per year. Classification counts made at automatic traffic-recorder stations on primary roads throughout the country showed an increase in the percentage for combinations, averaging 6 percent a year from 1939 to 1941, though there was a slight dip in the 1940 percentage. Replies to telegrams of May 19, 1942, relative to wartime traffic trends, indicated a sharp increase in the percentage for combinations in 10 of the 11 States submitting information on this point, but in only 3 States did the data go back of 1940. At one point in Connecticut the increase from 1939 to 1942 was 12.7 percent a year; at one point in Maryland the increase from 1937 to 1942 was 11.7 percent a year; and at four points in Colorado the average increase from 1937 to 1942 was 2.4 percent a year. It is probable that much of the increase in Connecticut and Maryland took place between 1940 and 1942, since data from other States indicate that the use of combinations was increasing rapidly during that period.

Considering the data in the preceding paragraph, an increase in percentage for combinations of 4 percent a year was decided upon as conservative. In other words, percentages for 1939, 1938, 1937, and 1936 were multiplied by 1.04, 1.08, 1.12, and 1.16, respectively, to obtain the 1940 estimates.

Percentage of loaded trucks.—Information on loaded vehicles as a percentage of all vehicles was obtained only at weight stations and these percentages were used exclusively in making the estimates. A special study of trucking on local roads in Michigan showed the ratio between percentages of loaded trucks and combinations on local roads and on main roads to be 0.97. The difference in percentages was regarded as too small to be significant. In the absence of other information bearing on this relation, the percentage of loaded vehicles as determined from the loadometer sample for each type of vehicle, was regarded as being applicable to local as well as main roads. When vehicles of all types were combined, this resulted in a percentage of loaded vehicles on local roads 98.5 percent of that on main roads.

Extensive loadometer resurveys were conducted in Kansas and Iowa in 1940, and these surveys showed an increase in percentage of loaded vehicles over that shown by the 1936 survey, averaging 2.4 percent per year for the two States. Kentucky estimated a decline in percentage of loaded vehicles of 1.4 percent per year from 1937 to 1941, and an 8-hour operation at a station on U S 1 in Maryland showed an increase of 2.0 percent per year from 1937 to 1942. Only the Iowa and Maryland data showed combinations separate from single-unit trucks, and in both cases all of the increase in percentage of loaded vehicles took place in single-unit trucks with a slight decrease, in fact, in the percentage of loaded combinations. The Iowa and Kansas data, being based on extensive surveys, should be much more reliable than the other data, but because of possibilities of sectional variations, the increase shown in those two States was reduced from 2.4 percent to 1.3 percent per year by averaging with the Kentucky and Maryland data. This is equivalent to 1.5 percent increase for single-unit trucks alone, and this figure was used in making estimates for these vehicles. It was assumed that there was no change in the figure for combinations. In other words, if the survey was made in 1936, the percentage of loaded single-unit trucks was multiplied by 1.06 to obtain the 1940 estimate, while the 1936 figure for percentage of loaded combinations was used without change in making the 1940 estimates.

Average carried load. Loadometer tables L–71, L–72, and L–73 show the average carried load (average load of loaded vehicles) for single-unit trucks, tractor-trucks and semitrailers, and trailers, respectively. The sample in these tables is necessarily limited to loaded vehicles for which the empty weight is known, or those for which the weight of the load carried is known by the driver or can be estimated accurately. In some States (such as Ohio) the empty weight of practically all vehicles is known because the law requires it to be stenciled on the side of the vehicle or shown on the registration card. The loadometer tables from such States include nearly all of the loaded trucks weighed. In some States, estimates of carried load were freely made, while in others carried loads were computed only in case the empty vehicle had previously been weighed and a sticker indicating the weight attached. For some reason the vehicles with known empty weight were heavier, on the average, than those with unknown empty weight. This is proved by the fact that in most States, the average total weight of loaded vehicles for which empty weight is known, taken from tables L–71, L–72, and L–73, is materially greater than the average total weight for all loaded vehicles taken from tables L–23, L–25, and L–27. This may be because more of the heavier vehicles are apt to pass a loadometer station both when empty and when loaded, or it may be because drivers were more likely to know the weight of heavier loads. Whatever the reason, the figures themselves show that the loaded vehicles included in tables L–71, L–72, and L–73 were heavier than the average and it may reasonably be assumed that their carried loads were also heavier.

In preparing the estimates, it was assumed that the empty weights shown in tables L–71, L–72, and L–73 were applicable only to the portion of the loaded vehicles shown in those tables, and that the empty weights of empty vehicles shown in tables L–24, L–26, and L–28 were applicable to the remainder of the loaded vehicles. An average empty weight was calculated for each type of vehicle by weighting in this manner, and was subtracted from the average weight of loaded vehicle of the same type, shown in tables L–23, L–25, or L–27, to determine the average carried load. This method of calculation gives a lower value for average carried load than that shown in tables L–71, L–72, and L–73 and it also gives a lower value than would be obtained by subtracting the average empty weight of all empty vehicles from the average total weight of all loaded vehicles. It is believed to be more accurate than either of the other methods.

The Michigan blanket-count tables for local roads show the average load carried by single-unit trucks on local roads to be 17.5 percent smaller than the figure for main roads as shown in loadometer tables. The average load carried by combinations was 21.8 percent smaller on local roads than on main roads. This difference in carried load is in addition to that which results from the smaller relative frequency of combinations on local roads. The survey in which the data on local roads were obtained was conducted from May to October 1936, and all weights were based on estimates made by the vehicle operators. Therefore the data are not strictly comparable with those for main roads in the regular loadometer tables, but the relations indicated are believed to be significant and approximately correct for Michigan conditions, at least. In other States the average load carried by single-unit trucks and that carried by combinations on local roads was assumed to be 80 percent of that carried by vehicles of the same type on primary roads.

As previously stated, Iowa and Kansas conducted extensive loadometer surveys in 1940, and data from these surveys, together with data from the original loadometer surveys of 1936, give the means of establishing the trends in weights in those States from 1936 to 1940. Because of the difficulties in computing carried load discussed above, and the relative smallness of the sample of vehicles with known carried load, it is possible that apparent trends in carried load may be due in part to differences in field and office procedures. Trends in total weight can be established from the data with much greater assurance of accuracy, and it is reasonable to assume that percentage increases in carried load have been at least as great as those in total weight.

In both Iowa and Kansas, the total weights of loaded single-unit trucks and of loaded combinations increased during the 1936–1940 period. The average increase per year for the two States was 5.4 percent for single-unit trucks and 1.0 percent for combinations. Results of an 8-hour operation at a station on U S 1 in Maryland in 1937 and again in 1942, indicated an average annual increase in total weight for the period of 2.8 percent for single-unit trucks, and also for combinations. Similar operations at a station on U S 5 in Connecticut in 1939 and 1942, indicated an average annual increase in total weight for the period of 0.5 percent for single-unit trucks and 2.3 percent for combinations. Data from 11 stations in Connecticut operated in 1934 and in 1940 showed a much larger rate of increase, but there is considerable question, in this case, concerning comparability. While the Iowa and Kansas data should be much more accurate than the others, they might be subject to sectional variation and for this reason were modified on the basis of the indications in Maryland and Delaware. The annual rates of increase in carried load during the 1936–39 period were therefore assumed to be 3.5 percent for single-unit trucks and 2.0 percent for combinations.

RURAL AND URBAN ORIGIN AND DESTINATION TABLES

Sources of data on vehicle-mileage. Highway planning survey table OD–4 shows for rural roads the truck-miles and percentages with rural origin and rural destination, with rural origin and urban destination or vice versa, and with urban origin and urban destination, in the survey year. This information is shown for a primary system, for a secondary system, and in some cases for a tertiary system also.

Usable OD–4 tables were received from the following 18 States: Idaho, Indiana, Iowa, Michigan, Minnesota, Nebraska, Nevada, New Hampshire, New Mexico, Ohio, Oregon, South Carolina, South Dakota, Tennessee, Utah, West Virginia, Wisconsin, and Wyoming (secondary system only). For all of these States except Oregon, the mileage classed as "primary" in the OD tables was the same as that classed as "main" in the ton-mileage tables, and that classed as "secondary" and "tertiary" (if any) was the same as that classed as "local" in the ton-mileage tables. In the case of Oregon, the small mileage classed as secondary was included with that of main roads in the ton-mile tables because its characteristics appeared to resemble those of main roads more nearly than those of local roads.

Information on origin and destination was obtained in the loadometer survey and these data were utilized in preparing the OD tables. In fact the information in the OD tables for trucks on the primary system was obtained in large part from the loadometer data and percentages from loadometer table L–4 therefore agree very closely with percentages for the primary system from table OD–4. In addition to the 17 States submitting usable OD tables for the primary system, the following 23 States submitted loadometer table L–4, permitting the calculation of origin and destination percentages for main roads: Alabama, Arizona, California, Colorado, Florida, Illinois, Kansas, Kentucky, Louisiana, Maryland, Massachusetts, Mississippi, Missouri, Montana, North Carolina, North Dakota, Oklahoma, Pennsylvania, Rhode Island, Texas, Virginia, Washington, and Wyoming. Origin and destination data were therefore available for main roads in 40 States and for local roads in 18 States.

Calculation of vehicle-mileage.—For the 18 States for which local road data were available, there was found to be an approximate relation between average traffic density on local roads (table 1 of the ton-mileage series) and the percentage distribution of vehicle-mileage according to origin-destination classification. This is not surprising, since a high average traffic density and a high proportion of travel with urban origin or destination are both caused by the presence of numerous cities of considerable size in a State. In figure 30, the average daily traffic density on local roads was plotted as abscissas, and the percentages with origin and destination both urban as ordinates for one curve, and the percentages with either an urban origin or destination or both as ordinates for a second curve. Lines were located through the two groups of points by the method of least squares. For 20 of the 22 States supplying origin and destination data for main roads, but not for local roads, the percentages applying to local roads were read from the curves. The average daily traffic density on local roads for the State was obtained from table 1, and a vertical line was drawn through this average density in figure 30. From the point where this vertical line intersected the lower curve, a horizontal line was drawn to the left, and the percentage with origin and destination both urban was read from the vertical scale at the left; and from the point where the vertical line intersected the upper curve, a horizontal line was drawn to the right and the percentage with origin and destination both rural was read from the vertical scale at the right. The difference between the sum of these two percentages and 100 was the percentage of urban-rural or rural-urban movement. The average daily traffic on local roads in Massachusetts and Rhode Island fell well outside of the limits of figure 30, and the percentages were estimated on the basis of corresponding percentages in other States similarly situated as regards the distribution of urban and rural population.

The percentages in table 6 relating to main roads were taken from table OD-4 when available from the State, and otherwise from table L-4. The percentages in table 7 relating to local roads were taken from table OD-4 when available from the State, and otherwise from the curves of figure 30 as explained above. In the case of the eight States for which no origin and destination data were available, the percentages were estimated on the basis of corresponding percentages in other States similarly situated as regards the distribution of urban and rural population. These eight States are not shown separately in the tables, however, but are classified as "Other States" in a single group. The truck-mileages for each system were taken from table 2 and were multiplied by the percentages to obtain the vehicle-mileage with origin and destination both rural, one rural and the other urban, and both urban. Table 8 for all rural roads was calculated by combining the figures in table 7 with those in table 6.

In the calculation described above the assumption is inherent that the proportions of the travel in the various origin-destination classifications did not change between the survey year and 1940. While there is no evidence on this point, it seems reasonable to assume that these relations remain stable, or change very slowly, except under very abnormal conditions such as have come about subsequent to 1940.

Calculation of ton-mileage.—The proportion of loaded vehicles in total truck traffic with origin and destination both urban, with one urban and the other rural, and with both rural, can be calculated on a percentage basis from loadometer table L-4, and an average carried load for vehicles in each of these three origin-destination categories can be calculated from table L-7. These tables were submitted by 39 States, these being the States named above as having submitted origin and destination vehicle-mileage data with the exception of Oregon. The data by States are shown in tables 6, 7, and 8.

For each State, the percentage loaded of trucks in each origin-destination category (urban-urban, rural-urban and urban-rural, and rural-rural) was calculated from table L-4, and a percentage of loaded vehicles in all categories combined was calculated separately for main roads and for local roads by weighting in accordance with the proportion of the vehicle-mileage in each origin and destination category on each system as shown in tables 6 and 7. The percentage of loaded vehicles on main roads was then adjusted to agree with that shown for the year 1940 in table 3, and the percentages in each origin-destination category were adjusted proportionately. The percentage of loaded vehicles on local roads was similarly adjusted to agree with the 1940 percentage shown in table 4. The average carried load for each origin-destination category was calculated from table L-7 and adjusted to the average carried load shown in tables 3 and 4 in the same manner that the percentage of loaded vehicles was adjusted.

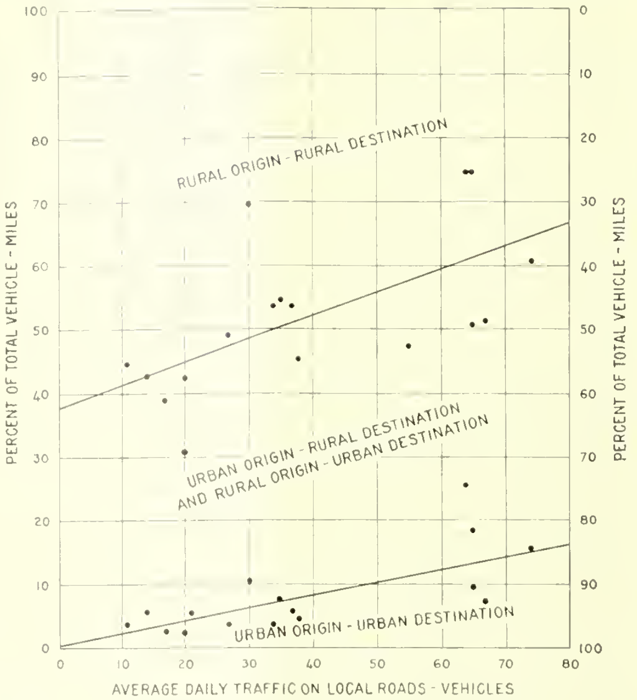

Figure 30.—Percentage of Total Vehicle-Miles With Origin and Destination Both Urban, One Urban and the Other Rural, and Both Rural in Relation to Average Daily Traffic on Local Rural Roads.

The calculations described in the preceding paragraph involve assumptions, as follows:

1. That the relations of the percentage of loaded vehicles in the three origin-destination categories are the same for local roads as for main roads.
2. That the relations of the averages for carried load for the three origin-destination categories are the same for local roads as for main roads.
3. That the relations of the percentages of loaded vehicles for the three origin-destination categories did not change on either system (main or local) between the survey year and 1940.
4. That the relations of the averages for carried load for the three origin-destination categories did not change on either system between the survey year and 1940.

It should be noted that the above assumptions apply only to relations of values for the three origin-destination categories and not to the values themselves. Thus, different proportions of urban-urban, rural-urban and urban-rural, and rural-rural travel on the two systems result in different values for percentage of loaded vehicles and for average carried load, even before adjustment to agree with tables 3 and 4. Likewise, changes in both percentage of loaded vehicles and average carried load between the survey year and 1940 on both systems are recognized, but the changes are assumed to have taken place to the same degree in each of the three origin-destination categories.

The ton-mileages shown in tables 9 and 10 for each system and for each origin-destination category were calculated by multiplying the corresponding vehicle-mileages shown in tables 6 and 7 by the percentage of loaded vehicles, and then by the average carried load. Table 11, for all rural roads, was calculated by combining the figures in table 10 with those in table 9.

TABLES SHOWING EXTENT OF TRIPS CLASSIFIED AS INTRASTATE, INTERSTATE, AND TRANSSTATE, AND CLASSIFIED ACCORDING TO NUMBER OF COUNTIES TRAVERSED

Sources of data on vehicle-mileage.—Highway planning survey table OD–2 shows the truck-mileage and percentages for intrastate, interstate, and transstate trips, on rural roads, subdivided into either two or three road system classifications, in the survey year. Table OD–3 shows the same information concerning trips involving travel in one county only, two counties, and three or more counties. These OD tables were available for the following 17 States: Idaho, Indiana, Iowa, Michigan, Minnesota, Nebraska, Nevada, New Hampshire, New Mexico, Ohio, Oregon, South Carolina, South Dakota, Tennessee, Utah, West Virginia, and Wisconsin. In addition, Wyoming tables for the secondary system were available. For all of these States except Oregon, the "primary" system classification in the OD tables was the same as the "main" road classification in the ton-mileage tables, and the "secondary" classification, plus the "tertiary" classification, if any mileage was so classified, was the same as the "local road" classification in the ton-mileage tables. In Oregon, the small mileage classed as secondary was included with the main road mileage in the ton-mileage tables because its characteristics appeared to resemble the main road mileage more nearly than the local road mileage.

Information as to origin and destination obtained in the loadometer survey was used in preparing table OD–2. Most of the information in this table for trucks on the primary system was, in fact, obtained from the loadometer data, and percentages from loadometer table L–5 therefore agree very closely with percentages relating to the primary system from table OD–2. In addition to the 17 States submitting OD tables for the primary system, the following 24 States submitted loadometer table L–5, permitting the calculation of origin-destination percentages for main roads: Alabama, Arizona, California, Colorado, Florida, Illinois, Kansas, Kentucky, Louisiana, Maryland, Massachusetts, Mississippi, Missouri, Montana, North Carolina, North Dakota, Oklahoma, Pennsylvania, Rhode Island, Texas, Vermont, Virginia, Washington, and Wyoming. Data concerning the intrastate, interstate, and transstate classifications were therefore available for main roads in 41 States and for local roads in 18 States. Information concerning the extent of trips by the number of counties traversed was available for local roads in 18 States and for primary roads in 17 States.

Calculation of vehicle-mileage.—The percentage of rural traffic in a State that is of an interstate or transstate nature depends on many factors, among which are the size of the State, the location of urban centers of population, natural or artificial barriers to traffic, and other geographical, social, or economic conditions. In the information submitted from the 17 States, for which origin-destination data were available for both main roads and local roads, only an approximate relation was found between the percentage of trucks classed as interstate or transstate on the main roads and that on the local roads. This approximate relation, however, is satisfactory for estimating the amount of interstate or transstate traffic on local roads in other States since the total truck traffic of the two types seldom exceeds 5 percent of the total traffic on these roads. In these 17 States, the amount of interstate traffic on local roads averaged 3.5 percent of the total traffic and the transstate traffic averaged 0.2 percent. The ratio of the percentage of interstate traffic on local roads to the percentage on main roads was 0.18 while the similar ratio for transstate traffic was 0.04. These relations between the percentages interstate and transstate of traffic on main and on local roads were used in obtaining an estimate of the percentages for these two types of traffic in 23 States which had compiled the loadometer tables but from which OD tables were not available.

The percentages in table 12 relating to main roads were taken from table OD–2 when available for the State and otherwise from table L–5. In the case of the seven States for which origin-destination data were not available, the percentages were estimated on the basis of corresponding percentages in other States similarly situated as regards size, location, and other pertinent factors. These seven States are not shown separately in the table but are classified as "Other States" in a single group.

The percentages for local roads were taken from table OD–2 when available for a State, and otherwise were estimated from the corresponding percentages for main roads, by multiplying by 0.18 in the case of interstate traffic and 0.04 in the case of the transstate traffic, as explained above. In table 13, only those States for which table OD–2 was available were listed separately and the States for which the percentages were estimated were lumped together in the single classification "Other States." The fact that the percentages on the line giving the subtotal for the States listed are the same as those on the line for "Other States" is accidental, in this case, as the method of making the estimates did not involve the assumption that these percentages would be the same.

Table 14 was prepared by combining the vehicle-mileage figures for main roads in each State, shown in table 12, with those for local roads, shown in table 13 or estimated as described. The reason the estimated figures for local roads were used State by State in table 14 for all rural roads, but not in table 13 for local roads, is that an error of high percentage in a local road figure would introduce only an error of low percentage in the corresponding figure for all rural roads.

The figures for total truck mileage in tables 12, 13, and 14, were taken from table 2. Vehicle-mileages of intrastate, interstate, and transstate truck traffic were calculated by multiplying the total truck mileages by the percentages.

The percentages of truck-miles on rural roads for trips which had an extent of one county only, of two counties, and of three or more counties, were taken from the OD-3 tables submitted by 17 States. Percentages for main rural roads and for local rural roads were obtained separately from the data for primary, secondary, and tertiary (if any) roads as described above. These percentages were applied to the truck-mileage figures for main roads and for local roads, given in table 2, to obtain the vehicle-mileages of trips requiring travel in one county, in two counties, and in three or more counties which are listed in table 18. In the last portion of table 18 the vehicle-miles of travel are combined for all rural roads and a percentage breakdown obtained from these total figures. Data were not available concerning the extent of trip, in terms of counties traversed, on either main or local roads, for the 31 States not submitting OD tables. Estimates were not made as there was no satisfactory basis for them.

The average area of counties, shown for each State in the last column of table 18, was computed from 1940 census figures, by dividing the land area of the State by the number of counties in the State. The purpose of this information is to aid in interpreting the data in terms of trip length.

In the calculations described above the assumption is made that the proportion of the travel that is intrastate, interstate, or transstate, or within one, two, or three or more counties, did not change between the survey year and 1940. While there is no evidence on this point, it seems reasonable to assume that these relations remain stable, or change very slowly, except under very abnormal conditions such as have come about subsequent to 1940.

Vehicle-mileage of loaded vehicles.—Figures for loaded vehicles, as a percentage of all vehicles engaged in rural intrastate, interstate, and transstate traffic were calculated from the data in loadometer table L-5. Such figures are hereafter referred to as "percentage of loaded vehicles." Table L-5 was submitted by 39 States, these being all of the States which are named above as having submitted some type of origin and destination information except Oregon and Vermont. The percentage of loaded vehicles for each of these categories was estimated for the remaining States, on the basis of the percentages in States similarly situated, and the estimated vehicle-mileage of loaded vehicles was obtained by multiplying the vehicle-mileage of all trucks by the percentage of those loaded for each category being considered. These latter vehicle-mileage figures (for loaded vehicles only) were then adjusted to agree with the total vehicle-mileage of loaded vehicles for 1940 in each State which previously had been established for each of the two systems of rural roads. The adjusted percentages of loaded vehicles by States are given in table 15 for main roads in 39 States and in table 16 for local roads in 17 States. The estimates of vehicle-mileage of loaded vehicles on main and on local roads were combined for 39 States and the percentages of vehicles loaded on all rural roads are tabulated in table 17. The data for States not tabulated separately are combined in each table.

The calculations described in the preceding paragraph involve two assumptions: First, that the relations of the percentage of loaded vehicles in the three origin-destination categories are the same for local roads as for main roads; second, that the relations of the percentage of loaded vehicles for the three origin-destination categories did not change on either system (main or local) between the survey year and 1940.

These assumptions pertain only to the relations of values for the three categories being considered and not to the values themselves. Thus different proportions of intrastate, interstate, or transstate traffic on the two systems result in different values for percentage of loaded vehicles even before adjustment to agree with previously established total values. Likewise, changes in percentage of loaded vehicles between the survey year and 1940 on both systems are recognized, but the changes are assumed to have taken place to the same degree in each of the three origin and destination categories.

Since the loadometer tables do not give any information concerning trip extent on the basis of the number of counties traversed, there is no direct means of determining separately the percentage of loaded vehicles making trips in one county, two counties, and in three or more counties. These percentages were estimated by assuming that they varied with the length of the trip and by utilizing the loadometer data concerning intrastate, interstate, and transstate trucking.

It was found from the loadometer data that the highest percentage of loaded vehicles was for the transstate trucks, the next highest for the interstate, and the lowest for the intrastate trucks. It was assumed that within each of these categories, the percentage of loaded vehicles increased as the number of counties traversed increased. For example, vehicles in the one-State-one-county, the one-State-two-counties, and the one-State-three-or-more-counties classifications shown in table OD-2 were assumed to have percentages of loaded vehicles in ascending order, with the total of these percentages the same as that shown in loadometer table L-5 for the intrastate classification.

The curve in figure 31, for main roads in the State of Iowa, illustrates the manner in which these estimates of percentage of loaded vehicles were made. The percentage of the total vehicle-miles classified as intrastate shown in table 12 is 74.8, and a vertical line was drawn at 74.8 percent, through D in the chart; the percentage classified as interstate is 20.2 and a vertical line was drawn at 95.0 percent (74.8 plus 20.2), through G on the chart. The remaining 5.0 percent is transstate traffic. The two vertical lines divide the chart into three sections representing intrastate, interstate, and transstate traffic, respectively. The percentage of loaded vehicles in each of the three classifications, shown in table 15, was then plotted at the center of the section, and sloping lines were drawn through these three points to form the curve shown in the figure. The slopes of these lines are, of course, indeterminate but when one is assumed, the other two are determinable. The slope of the line in the interstate field of the chart was generally the one assumed. Two penciled lines were first drawn lightly through the plotted point in this field, one passing through the plotted point in the intrastate field, and the other passing through the one in the transstate field. The line for the interstate field was then drawn so that it approximately bisected the angle between the other two lines.

The intrastate traffic is, of course, one-State traffic and is shown in table OD-3 as one-State-one-county, one-State-two-counties, and one-State-three-or-more-counties traffic. The intrastate field on the chart (fig. 31) was divided into one-county, two-counties, and three-or-more-counties fields, in accordance with the percentages shown for each in table OD–3, by vertical lines drawn through B and C in the chart.

The two-States-two-counties and the two-States-three-or-more-counties traffic is interstate, and vertical lines were drawn through E and F in the chart, in accordance with the percentages shown for these two classifications in table OD–3. The remaining percentage of the traffic is three-or-more-States-three-or-more-counties, and a portion of this is interstate and a portion is transstate. It is for this reason that the vertical line through F lies to the left of the vertical line through G in the chart.

The weighted averages of the percentages of loaded vehicles for one-county, two-counties, and three-or-more-counties traffic were then calculated from the chart as follows:

$$\text{One-county} = \frac{IA+JB}{2}$$

$$\text{Two-counties} = \left(\frac{JB+KC}{2}\times JK + \frac{LD+ME}{2}\times LM\right) \div (JK+LM)$$

$$\text{Three-or-more-counties} = \left(\frac{KC+LD}{2}\times KL + \frac{ME+OG}{2}\times MO + \frac{OG+PH}{2}\times OP\right) \div (KL+MO+OP)$$

Where necessary, small adjustments were made in the values thus computed to make them consistent with the percentage of loaded vehicles for the system as a whole, shown in tables 3 and 4. While some assumptions are involved in the method described above, it is believed that the resulting percentages are reasonably close approximations. The adjusted percentages of loaded vehicles in each of the three categories (one-county, two-counties, and three-or-more-counties) are shown for main roads in table 19. Values estimated similarly for local roads are shown in table 20. The vehicle-mileage data were combined for the two systems and the percentages of loaded vehicles derived for all rural roads are shown in table 21.

Ton-mile calculations.—The average load of loaded vehicles engaged in intrastate, interstate, and transstate traffic can be calculated from table L–9. These tables were submitted by 39 States, these being all of the States listed above as submitting data on percentage of loaded vehicles (table L–5). The average carried load for each origin and destination category was calculated from the table mentioned and these figures were adjusted to the total ton-miles of carried load shown in tables 3 and 4 in the same manner that the percentage of loaded vehicles was adjusted. The average carried loads and the ton-mileages carried on main roads are shown for 39 States and in a separate group for "Other States" in table 15. The same information for local roads is shown for 17 States in table 16. Vehicle-mileages of loaded vehicles and ton-mileage estimates for main roads and local roads were combined and average carried load and ton-mileage figures calculated for all rural roads, and these are shown in table 17.

The calculations described in the preceding paragraph involved certain assumptions, similar to those made concerning average percentage of loaded vehicles which were discussed above, and which are as follows: First, that the relation of the average carried loads for the three origin-destination categories is the same for local roads as for main roads; second, that the relation of the average carried loads for the three origin-destination categories did not change on either system between the survey year and 1940. As pointed out in the section concerning percentage of loaded vehicles, these assumptions have only to do with the relations of values for the three categories and not with the values themselves.

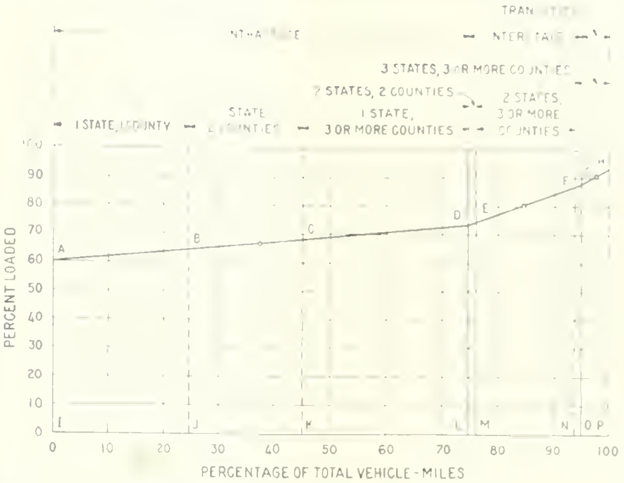

FIGURE 31.—CHART FOR IOWA MAIN ROADS, ILLUSTRATING METHOD OF ESTIMATING THE PERCENTAGE OF LOADED TRUCKS FOR ONE-COUNTY, TWO-COUNTY, AND THREE-OR-MORE-COUNTY TRUCK TRAFFIC.

The average load carried by trucks making trips having an extent of one county only, two counties, and three or more counties was calculated using the same plan as that used in calculating the percentage of loaded vehicles of traffic in these three categories. A graph was drawn for average carried load similar to the one for percentage of loaded vehicles in figure 31, using the average carried load in tons for intrastate, interstate, and transstate traffic. In this case, however, the horizontal scale represented loaded vehicle-mileage instead of total vehicle-mileage. Such adjustments were made as were necessary to make the average carried load for each system agree with figures previously determined and shown in tables 3 and 4 of the ton-mileage series. The averages for carried load and for ton-mileage of carried load on main roads are shown for the 16 States from which the complete information was available in table 19 and for local roads in table 20. The loaded vehicle-mileage data used in calculating tables 19 and 20 and the ton-mileage figures tabulated in these tables were added together and these figures used in calculating the average carried load in tons for all rural roads, shown in table 21.

WARTIME CHANGES

All of the States except North Carolina undertook weighing operations in 1942, and submitted tables of the "W" series showing wartime trends. In New York the survey was postponed until fall because the only earlier weight data that could be used for comparisons were collected in the fall of 1939. Although the New York tables of the "W" series have now been received, they were not available when the tables and charts were prepared in final form. Since the trends shown by the New York tables are very similar to the trends calculated for the region on the basis of data from Pennsylvania and New Jersey, the tables and charts were not revised to include New York. They are based on data from all States except New York and North Carolina.

The general procedure used in calculating the figures for 1942 in table 22 involved the following steps:

a. The calculation, for each State, of trends from the original survey year to 1942 on the basis of data taken at the same stations in comparable periods in the two years. For example, it may have been found that, at the ten stations operated in a State in the 1942 survey, 27.5 percent of all vehicles counted were trucks while, at the same stations, in a comparable summer period in 1937, only 19.1 percent were trucks. The 1942:1937 trend ratio for this item would then be 27.5÷19.1=1.44.

b. The calculation of trends from 1940 to 1942 by dividing the trends from the survey year to 1942 by the trends from the survey year to 1940. Thus, continuing the above example, assume that the 1940:1937 trend ratio for this item, used in computing table 2, was 1.08. The 1942:1940 trend ratio would then be 1.44÷1.08=1.33.

c. The calculation of values for each item for the full year 1942 by multiplying the 1940 value shown in table 2 or 3 by the 1942:1940 trend ratio for the item. In the example, assume that table 2 shows that, for the State in question, 21.3 percent of the annual vehicle-mileage in 1940 was by trucks and truck combinations. The corresponding value for 1942 would be 21.3×1.33=28.3 percent.

The following items were calculated for 1942 by means of trend ratios as above described:

1. Traffic by all vehicles.
2. Trucks as a percentage of all vehicles.
3. Combinations as a percentage of all trucks.
4. Percentage of combinations loaded.
5. Average carried load for combinations.
6. Single-unit trucks as a percentage of all trucks.
7. Percentage of single-unit trucks loaded.
8. Average carried load for single-unit trucks.

These items were calculated for each State separately, and combined by regions for inclusion in table 22. Some reservations must be made with respect to items 5 and 8. In the 1942 survey those vehicles weighed while empty were not the same vehicles as those weighed while loaded. Since the percentage of travel while empty is greater for certain types of vehicles than for others, the sample of empty vehicles was not strictly comparable with the sample of loaded vehicles. In this short survey, it was not possible to weigh the same vehicle empty on one occasion and loaded on another, and therefore trends in carried load could not be computed directly. These trends were estimated on the assumption that fluctuations in carried load were proportional to fluctuations in gross load. This assumption did not tend to exaggerate changes, and may have tended to minimize them to some extent.

As regards item 1, ratios for traffic by all vehicles established by the summer counts were not believed to be representative of the entire year. In normal years there is a large tourist traffic in the summer. War restrictions have been aimed particularly at recreational driving and have checked it to a much greater extent than the more essential driving which is characterized by less seasonal fluctuation. Subsequent to the initial compilation of the table, data became available from automatic traffic recorders operated continuously throughout 1940 and 1942 at 466 stations in 46 States. As had been expected, these data showed that the 1942 vehicle-mileage computed on the basis of the midsummer trends was too low, and the column showing vehicle-miles by all vehicles was revised on the basis of the trends shown by the automatic traffic recorder data. The vehicle-miles were computed for each State separately and then combined to obtain the regional and United States totals shown. For the two States without complete automatic traffic recorder data (Delaware and New Jersey) 1942:1940 ratios were estimated on the basis of ratios in nearby States with similar traffic conditions.

Truck traffic is not subject to the same seasonal fluctuations as passenger-car traffic, and the truck-mileage trends determined from the midsummer counts might therefore be expected to be much more nearly representative of the ratios for the entire year than those for passenger vehicles. The truck-mileages calculated from the data of the special survey were therefore left unchanged in table 22 and the percentage relations between truck miles and vehicle-miles by all vehicles were recalculated. The relations were then checked against relations established from continuing survey classification counts.

Classification counts were made in 1940 and 1942, either monthly or at least once in each season, at 122 stations (identical for all counts) in the following 16 States: California, Florida, Illinois, Kentucky, Louisiana, Montana, Nebraska, Nevada, Oklahoma, Pennsylvania, South Dakota, Texas, Utah, Washington, West Virginia, and Wisconsin. Generally the counts were made only on weekdays. It would be expected that trucks would constitute a higher percentage of total traffic than would be found if Sunday counts were included. The percentage was 22.5 in 1942 compared to 21.6 in 1940, the 1942:1940 ratio being 1.04, which is exactly the same as the ratio shown in table 22. If the percentages are applied to the vehicle-mileage by all vehicles, the 1942:1940 truck-mileage ratio from the continuing classification count data is 0.885 compared to 0.89 shown in table 22. This very close check based on data for a full year from 16 States, well distributed geographically, indicates that the ratios based on the data from the special midsummer survey are representative of the ratios for the entire year, so far as trucks are concerned.

SOME PROBLEMS OF ROAD CONSTRUCTION AND MAINTENANCE IN ALASKA

By STEPHEN TABER, Professor of Geology and Mineralogy, University of South Carolina

ALASKA has an area of 586,400 square miles, approximately one-fifth that of the United States, but in 1939 it had fewer than 2,500 miles of highway and only 1,600 miles of sled roads. This lack of appreciable road mileage is due chiefly to the sparseness of population though difficulties of road construction and maintenance are factors also. The population of Alaska, according to the 1940 census, was 72,524, including 32,458 Indians and Eskimos. Normally the population is augmented during the summer by several thousand laborers in gold mines and fish canneries. In the last 2 years an unreported number of defense workers and men of the armed forces have moved into the territory.

For several years the longest continuous road in Alaska has been the Richardson Highway from Valdez to Fairbanks with its extension, the Steese Highway, from Fairbanks to Circle, a total of 522 miles (fig. 1). The Richardson Highway has been kept open for traffic for about four-and-a-half months of the year, roughly from the first of June through September. Just prior to the present war a road was begun to connect Anchorage with the Richardson Highway. Except in towns, none of the roads in Alaska has been hard surfaced.

Alaska possesses a very wide range in topography, climate, and soil. Therefore the factors affecting road conditions vary greatly in different localities. The problems of highway construction and maintenance commonly encountered in the United States are found in Alaska together with additional problems which are unfamiliar to the majority of American engineers. Most of these new problems result from the freezing and thawing of water under conditions peculiar to subpolar regions.

Nearly 30 years ago the writer began an experimental investigation of the freezing of soils.[1] The experiments were later carried on with the financial cooperation of the Public Roads Administration, formerly the Bureau of Public Roads.[2] In 1935 the author, under the sponsorship of the Geological Survey, made a field study of problems connected with perennially frozen ground in Alaska.[3] This experience enabled him better to understand and appreciate the difficulties with which road engineers in a subpolar country have had to contend.

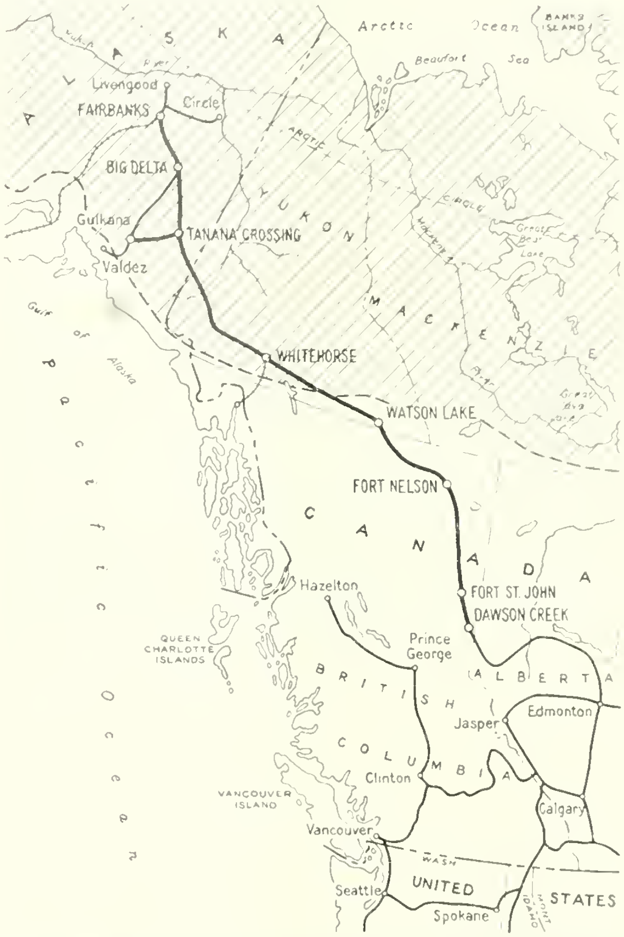

Figure 1.—Location of Alaska Highway and Connecting Roads. Area of Perennially Frozen Ground is Shown by Hatching.

THE PERENNIALLY FROZEN GROUND

Throughout most of Alaska, except for a broad coastal zone in the south and southeast (fig. 1), the subsoil is perennially frozen to depths which sometimes exceed 300 feet. The surface soil is subjected to seasonal thawing and refreezing to a depth of a few inches or a few feet depending upon climate, type of soil, amount of water content, exposure to sun, and amount of insulation by snow and vegetal cover.

In many localities the frozen ground contains masses of relatively pure ice occurring partly in almost horizontal layers or lenses, and partly in the form of nearly vertical wedge-shaped veins which, in places, merge into overlying ice layers (fig. 2). Layers vary from a few inches to 12 feet or more in thickness, and veins are as much as 8 feet in width. In some areas ice amounts to 80 percent of the volume of frozen ground.

The deep freezing occurred in early Pleistocene time, perhaps a million years ago. Since then, part of the ground has been deeply thawed and refrozen at least once but most of it has remained continuously frozen to the present time. There is considerable evidence that during the last few thousand years the climate, on the average, has become warmer and that the area of perennially frozen ground has been decreasing.

Over large areas the surface of Alaska is covered with a dense growth of mosses, grasses and other small plants which grade downward into a peaty layer. This plant

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[1] The Growth of Crystals Under External Pressure, The American Journal of Science, vol. XLI, June 1916; Ice Forming in Clay Soils Will Lift Surface Weights, Engineering News-Record, vol. 80, No. 6, February 7, 1918; Surface Heaving Caused by Segregation of Water Forming Ice Crystals, *ibid.*, vol. 81. No. 15, October 10, 1918.

[2] Frost Heaving, The Journal of Geology, vol. XXXVII, No. 5, July–August, 1929; The Mechanics of Frost Heaving, *ibid.*, vol. XXXVIII, No. 4, May–June 1930; Freezing and Thawing of Soils as Factors in the Destruction of Road Pavements, Public Roads, vol. 11, No. 6, August 1930.

[3] Perennially Frozen Ground in Alaska, Its Origin and History, by Stephen Taber, will be published shortly by the Geological Society of America.

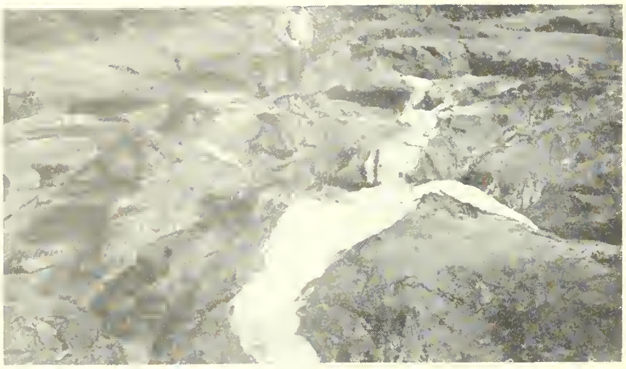


Figure 2.—Ice Veins in Silt Exposed During Stripping Operations on Engineer Creek, Near Fairbanks.

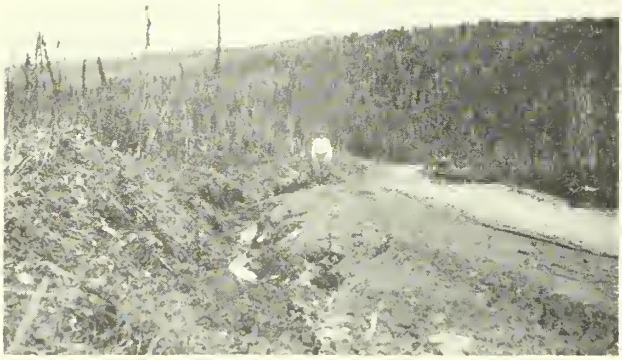
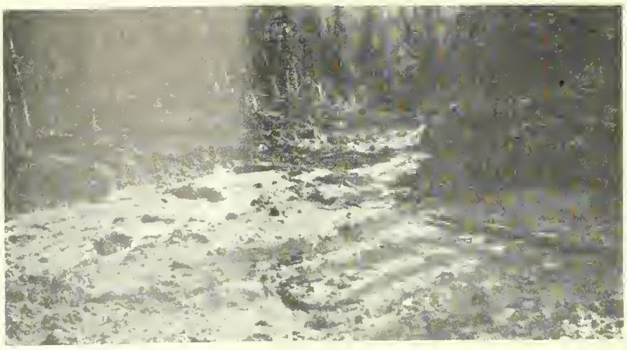
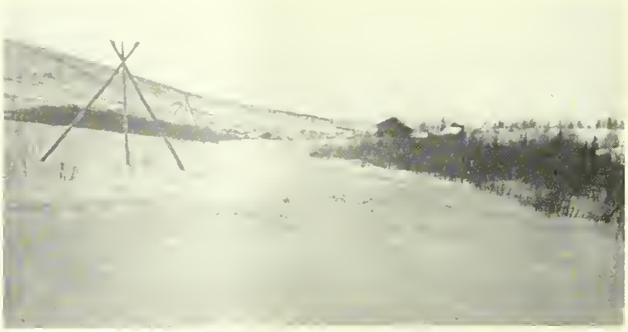


Figure 3.—Widening the Fairbanks-Livengood Road. Removal of Dense Cover of Vegetation and Peat Has Resulted in Deeper Summer Thawing of Perennially Frozen Ground.



By Courtesy of Alaska Road Commission.

Figure 4.—Fluid Mud Flow on Gulkana-Chisana Road.



By Courtesy of Alaska Road Commission.

Figure 5.—Ice on Steese Highway at Fox Gulch, March 14, 1933. Telephone Wires Are Strung on Tripods to Avoid Heaving. Little Snow Accumulated During the Winter.

material is like a sponge in its absorption of water and thus prevents rapid run-off. It also acts as an insulating blanket so that slow thawing through the summer helps to keep the soil wet. Most of the precipitation in central Alaska occurs in the form of light showers during summer and early fall. Since water does not percolate downward through the frozen subsoil, and surface drainage is slow, the soil is usually saturated close to the surface when seasonal freezing occurs. Removal of the vegetal cover in the construction of roads results in deeper thawing.

SEGREGATION OF WATER IN THE FORM OF ICE DURING THE FREEZING OF SOILS

Miniature ice layers and ice veins, similar in every respect except size to those found in Alaska, have been produced by the writer in laboratory experiments.

When soils saturated with water are cooled from the surface downward, as happens normally in the ground, the water immediately below the zone of freezing is usually free to move either upward to feed the growing ice crystals or downward away from them. In other words, the freezing occurs in an open rather than in a closed system, and any surface heaving, or other pressure effect observed, results from the growth of ice crystals rather than from the increase in volume that accompanies the freezing of water.

If freezing takes place in coarse-grained material, such as gravel and clean sand, ice forms in the relatively large interstitial spaces, the excess water being expelled downward so that little or no heaving of the surface occurs. But if freezing takes place in fine-grained soils, water is fed to the growing ice crystals which build up masses of relatively pure ground ice. The pressure developed by a growing ice crystal is in the direction of growth, which is usually normal to the cooling surface. Surface heaving results from the formation of masses of ice crystals and is approximately proportional to the size of these masses. Therefore, under favorable conditions, the uplift may greatly exceed that which could be brought about by a change in volume of the water when frozen. In some instances the surface uplift is equal to 80 percent or more of the depth of freezing.

Over large areas in central and northern Alaska the prevailing soil is a fine-grained silt containing little or no clay. This soil is, therefore, quite permeable and, when slowly cooled, it is a medium highly conducive to the formation of segregated ice. The silt ranges in thickness from shallow deposits to more than 200 feet in some of the valleys; and it is in these thick deposits that most of the very large masses of ground ice are found. The fine-grained silts are most abundant in the large areas of central and northern Alaska which were not covered by the continental ice caps of Pleistocene time, as were large areas farther south.

DIFFERENTIAL FROST HEAVING

Frost heaving is especially intense over perennially frozen ground because the fine-grained soil is usually saturated with water close to the surface when freezing occurs. As a result of repeated cycles of freezing and thawing, posts and similar objects work up out of the ground to an extent that is unknown in the northern parts of the United States. Partly for this reason, and partly because of the difficulty of digging deep holes in frozen ground, telegraph and telephone wires are commonly strung on tripods (figs. 5 and 7).

Uniform heaving has little direct effect on highways; but differential heaving of the surface may damage highways and even interrupt traffic. The principal causes of differential heaving of the surface are local differences in soil texture, the water supply, and the rate of freezing.

The most striking instance that has come to the writer's attention of the differential heaving of a roadway occurred in the early winter of 1934 on the Steese Highway at mile post 24, northeast of Fairbanks. The locality was examined the following summer by the writer in company with Mr. Frank Nash of the Alaska Road Commission, who described the heaving in detail. The road, which passes along the lower part of a steep hill slope having a southerly exposure, was locally heaved up with the formation of a hummock about 20 feet across and 7 to 8 feet high. A crack about 1 foot wide formed along the crest of the hummock.

The soil at the place where the hummock formed is a fine, micaceous silt, containing angular fragments of the mica schist from which it was derived by processes of disintegration, chiefly frost action. The surface is covered with a dense growth of mosses and grasses which grade downward into a foot or more of peaty material. This vegetal blanket had been removed locally in grading the road. A shallow ditch along the upper side of the road collected seepage water draining down the slope and conducted it to a culvert passing under the gravel-surfaced road.

Freezing, and therefore heaving, began first under the road which was not protected by the insulating blanket of vegetal material. The culvert, if not already blocked, was soon rendered useless by the heaving. For some time after heaving began, water continued to percolate down the slope under the protection of the vegetal mat which had been warmed by long exposure to the sun. Rupturing of the surface crust along the top of the growing hummock exposed the underlying soil to more rapid cooling. Such excessive differential heaving is, of course, very unusual. It may be prevented by a drainage system that will not allow water to accumulate under the road.

On slopes with southerly exposures summer thawing normally penetrates to a greater depth than on other slopes, and this helps to explain why frost troubles are somewhat more common where roads are located along the bases of slopes facing the south. When refreezing occurs in the fall and early winter the thaw-water, under favorable conditions, may continue for some time to percolate down the slope between the downward freezing surface layer and the deeply frozen ground below. If the percolating water becomes trapped between the frozen layers, so that it cannot readily escape, hydrostatic pressure may be developed. This condition tends to accentuate heaving. Occasionally the heaving ground is ruptured with a loud report like the sound of a cannon, and fragments of ice and frozen soil may be flung considerable distances. Such explosive effects are due to the sudden relief of stresses in elastic material. The stresses are set up by the slow growth of masses of ground ice or by the hydrostatic pressure of confined water or by both of these factors. Such occurrences are rather rare in Alaska, and they should not be confused with the relatively common tension cracks that form suddenly as a result of contraction of frozen ground in very cold weather. Cracks of this type are seldom more than one-half inch in width, and they tend to close as the temperature rises.

By Courtesy of Alaska Road Commission.

FIGURE 6. THICK ICE BURYING HIGHWAY IN FAIRBANKS DISTRICT, MARCH 14, 1933. LITTLE SNOW ACCUMULATED DURING THE WINTER.

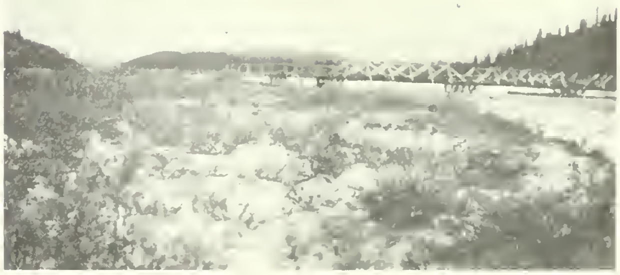


FIGURE 7. FOUR-SPAN BRIDGE OVER STREAM ON RICHARDSON HIGHWAY, SUMMER OF 1935. BRIDGE WAS BURIED UNDER ICE DURING THE PREVIOUS WINTER.

DOWN-SLOPE CREEP

As freezing progresses inward from a sloping surface, each ice crystal formed grows by additions of molecules to its base and pushes outward in the direction of heat conduction, or normal to the surface. The soil particles are moved outward in the same direction, those at the surface being displaced the most, and those near the bottom of the zone of freezing the least. On thawing, the soil settles vertically downward under the influence of gravity. Each freezing cycle, therefore, results in a small down-slope displacement of the surface. Thaw-water escaping through the expanded soil displaces some soil particles, and thus hastens downward creep. Because of the abnormally high water table on steep slopes where ground is perennially frozen, soil creep is relatively rapid. In Mount McKinley National Park, down-slope displacement of the road is reported by the road engineers to be especially rapid where it passes along the south-facing slope between the Savage and Sanctuary rivers.

PROBLEMS RESULTING FROM THAWING

Thawing penetrates to greater depth under a bare roadway than under an insulating blanket of peat and vegetation. A road constructed on perennially frozen ground containing a high percentage of ice may be passable for cars and trucks when newly graded, but become impassable later as a result of deep thawing. Where the slopes are steep, mud flows and slumps occur; where the ground is low and flat, the roadway becomes deep mud. The road from Fairbanks to Livengood, north of the Chatanika River, was graded in 1933, but some parts were in such bad condition in the summer of 1935 that the road was not open to

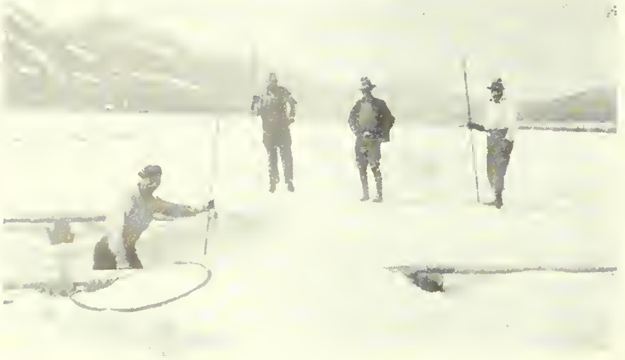


Photo by Mrs. H. E. Revelle.

Figure 8.—Bridge Buried Under Ice, Richardson Highway, Spring of 1935.

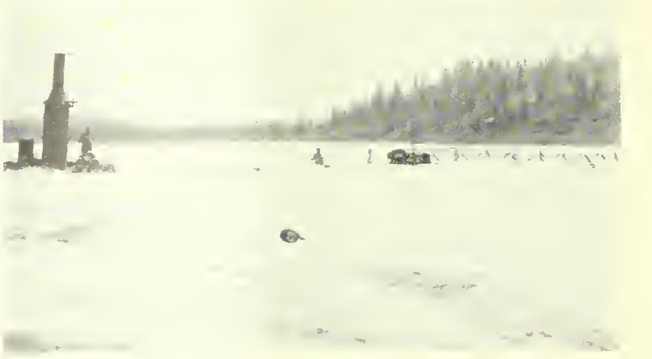


Photo by Mrs. H. E. Revelle.

Figure 9.—Removing Ice From Bridge on Richardson Highway. Boiler Furnishes Steam for Thawing. Little Snow on Valley Slopes.

traffic. In broadening the road the grading machine exposed a mass of ground ice at the point shown in figure 3.

Where drainage is good, so that thaw-water can escape, a road becomes stabilized in time but on flat, poorly drained surfaces it is often preferable to leave the insulating turf in place and build the road on top of it, or on top of a corduroy laid on the turf, so as to prevent thawing under the roadbed.

More water can be introduced into a soil in situ by freezing and thawing than in any other way. Introduction of water by percolation is limited by a soil's porosity but the volume of segregated ice formed in fine-grained soils may be several times the volume of the voids present before freezing because additional water has been drawn up from below. The thin layer of thawed soil resting on perennially frozen subsoil is commonly saturated with water when seasonal refreezing takes place. Freezing tends to concentrate the water in the upper part of the surface layer at the expense of water in the lower part and, where this water is replaced by down-slope percolation from a higher elevation, the total water content may be greatly increased locally as in the case of excessive heaving previously described. Rapid thawing of surface soils by warm rains in early summer often results in shallow mud flows of high fluidity as shown in figure 4. Interstitial water adds to the weight of the soil mass and also acts as a lubricant. Compaction due to settling or shearing of saturated silts that have been expanded by freezing causes them to behave like a liquid.

BURIAL OF HIGHWAYS UNDER ICE

Springs and seepages of ground water that continue to flow after most of the surface soil has frozen build up large deposits of ice (figs. 5 and 6) locally called "glaciers" which may cover an area of thousands of square yards and attain a thickness of more than 25 feet. This happens chiefly near the bases of steep slopes with southerly exposures. Similar accumulations of ice formed by streams that freeze to their beds and overflow are also erroneously called "glaciers." The ice tends to accumulate below points where streams emerge from under protective screens of vegetation or from under glaciers and where streams spreading out over gravelly beds become shallow.

In winter, the road in Mount McKinley National Park becomes buried at several places under such heavy deposits of ice that it is necessary to thaw and remove the ice to open the road for early summer traffic. The worst condition in recent years developed at a point where a corrugated, galvanized iron culvert conducted the water of a small stream under the road. The water, trickling down under the protection of a dense growth of vegetation, continued to flow after freezing began along the exposed roadbed. Rapid conduction of heat by the metal culvert hastened freezing and, when the culvert became blocked, water overflowed to freeze on the road and bury it under ice. Perhaps the trouble might have been prevented by substituting for the metal culvert a wooden culvert with steep slope and extending it well up into the vegetation so as to protect the water from rapid cooling while flowing quickly under the road. Wood may be used for culverts in Alaska because it decays very slowly under the climatic conditions prevailing in areas of perennially frozen ground.

Where the Richardson Highway passes through the valley of the Delta River, a four-span, wooden bridge (fig. 7) over a glacial stream was almost completely buried under ice during the winter of 1934–35. The stream continued to flow from under the protecting glacier during cold weather, but a short distance below, near the bridge, the water spreading out over its broad gravelly bed became cold enough to freeze. Ice forming on the bed of the stream caused the water to overflow. When the stream froze over, water broke through cracks to overflow and form ice farther down. Repetition of these processes resulted in the burial of the bridge as shown in figure 8.

The bridge was cleared for traffic in the spring by melting and breaking out blocks of ice which could be hauled away by a tractor (figs. 9 and 10). Steam from a boiler was used for the thawing. Trouble such as this can be prevented in some places by wise selection of a road's location or by the building of a relatively high bridge.

FLOODS

The annual precipitation over most of the area of perennially frozen ground in Alaska is less than 15 inches. At Fairbanks it is 11.03 inches. Surprisingly little snow accumulates in the winter in central Alaska since more than half of the precipitation is in the form of light rains, chiefly during July, August, and September. Under these conditions large floods in the rivers are rare except during the spring "break-up" when ice jams are common.

Flash floods occur on some of the small mountain streams, especially when warm summer rains cause rapid melting of glacial ice. Such floods are most troublesome in the valley of the Delta River where the

Richardson Highway crosses several alluvial cones. These cones have been built up by small streams during floods which bring down heavy loads of boulders, pebbles, and sand. Deposition of the material is caused by the sudden change in grade as the streams enter the broad valley of the Delta River. The road is commonly downgrade in both directions from the bridge over a stream that is building a cone.

When floods occur, small bridges are frequently washed out. Sometimes a stream takes a new course down a cone, leaving the bridge spanning a dry channel. In the summer of 1931 a small bridge was completely buried under a deposit of boulders brought down by Gunnysack Creek, making necessary the building of a new bridge (fig. 11).

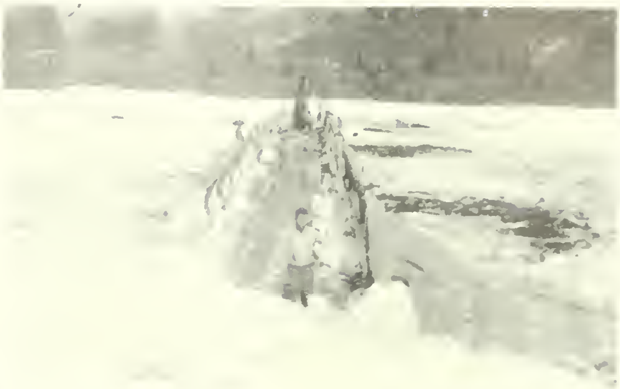


Photo by Mrs. H. E. Revelle.

FIGURE 10.—REMOVING ICE FROM BRIDGE ON RICHARDSON HIGHWAY, SPRING OF 1935.

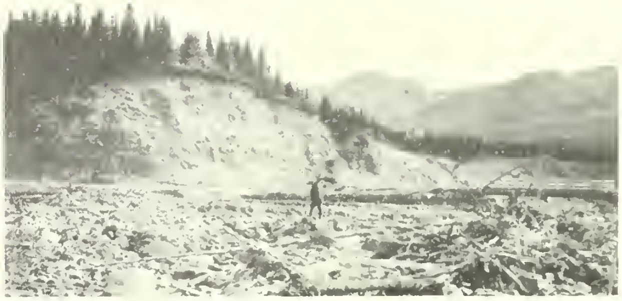


FIGURE 11.—BRIDGE OVER GUNNYSACK CREEK ON RICHARDSON HIGHWAY. AN EARLIER BRIDGE IS BURIED UNDER BOULDERS AT THE POINT WHERE MAN IS STANDING.

THE ALASKA HIGHWAY

The new road to Alaska, in passing northward from Dawson Creek, probably does not pass over perennially frozen ground until it crosses into Yukon Territory. In Alberta and British Columbia perennially frozen ground is found only locally and at high altitudes, but it is reported to extend over nearly all of Yukon Territory.[4] Muskegs, (sphagnum bogs with tussocks) are troublesome south of the region of perennially frozen ground where they thaw completely in summer, but the road has been so located as to avoid most of them.

Much of the country traversed by the Alaska highway was subjected to Pleistocene glaciation, and thick deposits of fine silts containing large amounts of ground ice are not common in the glaciated areas. The highway joins the Richardson Highway near the junction of the Delta and Tanana rivers and just north of the glacier section where maintenance has been so difficult. Glaciers are not found in the immediate vicinity of the new road. Along its route precipitation in the form of both rain and snow is relatively light.

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[4] Frozen Ground in the Glaciated Parts of Northern Canada, by W. A. Johnston, F. R. S. C., Transactions of the Royal Society of Canada, section IV, series III, vol. XXIV, May 1930.

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A SECTION OF THE THREE-STATE HIGHWAY

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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HIGHWAY LAND ACQUISITION COSTS AND PRACTICES IN ILLINOIS AND WISCONSIN

A STUDY OF THE THREE-STATE HIGHWAY

Reported by **DAVID R. LEVIN**, Transportation Economist

PUBLIC AUTHORITIES, particularly those concerned with highway transport, have known for some time that the construction of new highway facilities and the modernization of existing roads have been impeded by prohibitive costs of land acquisition and almost insurmountable legal and administrative obstacles. Yet until recently,[1] little organized effort had been directed toward the investigation and elimination of these difficulties which, in the aggregate, constitute the so-called right-of-way problem. In 1939, the Public Roads Administration undertook a brief investigation of the Three-State Highway,[2] with the dual objective of ascertaining highway right-of-way costs and revealing such land acquisition difficulties as may be expected to occur on a long stretch of highway traversing rural, suburban, and urban areas devoted to a variety of land uses. It is the purpose of this report to present the findings of this pioneer study.[3]

FACTS AND CONCLUSIONS SUMMARIZED

The problems involved in the construction of the Three-State Highway are typical of those encountered in any major highway improvement. The portion of the highway studied is located in six counties of which three are in Illinois and three are in Wisconsin. It traverses urban, suburban and rural territory representing a wide diversity of land uses.[4]

Mileages, areas, number of parcels.—As of September 1939, the right-of-way for 72 miles of a proposed 161-mile highway improvement had been obtained, involving 1,065 acres of land of which 83 percent was rural, 9 percent suburban, and 8 percent urban. A total of 835 parcels was acquired, averaging 11.6 parcels per mile of road. Of the total area acquired, 67 percent consisted of agricultural lands, 21 percent was residential, and the remaining 12 percent was devoted to other uses. Although commercial property represented but 2 percent of the area acquired, it accounted for 20 percent of the entire cost ($1,228,281) of the acquisitions.

Condemnation, donation.—In acquiring more than a thousand acres of land, 6 percent had to be condemned. Resort to court proceedings is likely to add a minimum of from $300 to $400 to the costs of a given acquisition. Donations were few, except in Milwaukee County, Wis., where the practice seems to prevail of offsetting the benefits accruing to property from an improvement against the value of land acquired, one invariably canceling the other.

Wisconsin award system.—The so-called statutory award system in Wisconsin seems to expedite the acquisition of lands and the construction of highways. The State or a county makes an award for the value of land and damages and tenders that amount to the property owner. Construction may commence immediately afterward. Illinois has no such summary procedure.

Total acquisition costs.—The average total cost of land acquisition per mile of the Three-State Highway was $17,124; of this total, 29 percent represents the cost of "bare" land, 69 percent the amount of damages, and 2 percent the out-of-pocket expenditures for services incidental to acquisition.

Population density.—Although only 8.7 percent of the area acquired was in suburban territory, 16.0 percent of the total cost of $1,228,281 was incurred therefor; 8.4 percent of the area, located in urban sections, was assigned 19.3 percent of the total cost; and 82.9 percent of the area was in rural places and accounted for 64.7 percent of the total cost.

Costs per mile and per frontage-mile.—The average cost per frontage-mile for all property acquired was $9,812. The average cost in rural areas was $7,580 per frontage-mile, in suburban areas almost twice as much, and in urban places more than four times the rural average. These frontage mileages translated into ordinary mileage costs indicate that the cost of acquiring property necessary for the Three-State Highway was approximately $15,000 per mile through rural areas, $29,000 per mile in suburban territory, and $67,000 per mile in urban places.

Costs of "bare" land.—The purchase of 718 acres of "bare" agricultural land entailed an expenditure of $90,763, or $126 per acre. A frequency distribution of unit land costs shows that the average land cost was $145 per acre for all farm lands, $774 per acre for residential lands in rural areas, and $2,033 per acre for residential lands in urban areas.

Damages and incidental fees.—Damage costs averaged $11,801 per mile of highway. Out-of-pocket incidental fees averaged $330 per mile of road.

Marginal land acquisition.—Forty-two parcels of marginal or so-called "excess" land, aggregating 25.6 acres, were purchased at an estimated total cost of $30,823. The parcels are largely in Du Page County, Ill. Of this land, 14.7 acres were traded for other lands needed for highway purposes, 0.6 of an acre was sold, 2.7 acres were transferred to other governmental agencies, and 7.6 acres still remain in county ownership. Some surplus land was purchased in order to avoid the payment of large amounts of consequential damage because of severance and destruction of plot-

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[1] The Defense Highway Act of 1941 authorizes the Public Roads Administration to extend Federal aid to the States in the acquisition of lands for the strategic system of highways, for access roads, for flight strips, and for off-street parking facilities.

[2] So called because the plans for ultimate development indicate it will traverse portions of Wisconsin, Illinois, and Indiana.

[3] The investigation was undertaken by the Public Roads Administration under a cooperative agreement with the University of Wisconsin. It was directed by Dr. Henry R. Trumbower, Senior Agricultural Transportation Economist, Public Roads Administration, and Professor of Transportation, University of Wisconsin, Madison, Wisconsin. H. R. Briggs, assisted by the author, supervised the project.

[4] See Glossary of Terms for definitions of technical words.

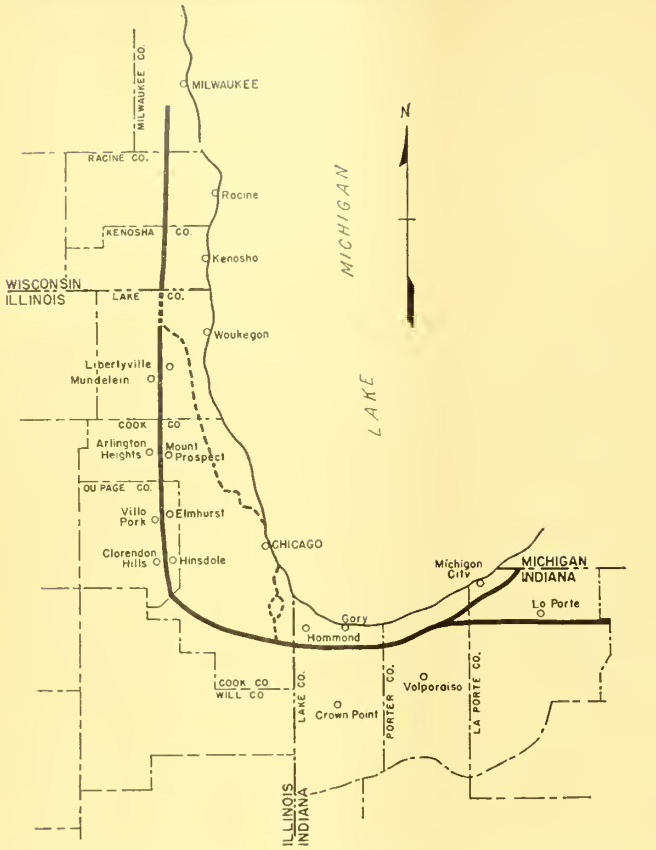

Figure 1.—The Three-State Highway as Originally Planned in 1925. Data Taken from Files of Chicago Regional Planning Association.

tage values. In general, the device of marginal land acquisition was used quite successfully in Du Page County.

Highway widening versus highway relocation.—Analysis of costs on several widened or relocated sections of the highway indicates that land acquisition costs for widening are considerably greater than for relocation. Kenosha County, Wis., and Du Page County, Ill., provide striking illustrations.

INVESTIGATION COVERED ALL PHASES OF LAND ACQUISITION COSTS

The Three-State Highway, when completed, is to be a through route extending from Milwaukee south and east around Chicago to a point somewhere in the eastern part of Indiana. It was conceived of as a connecting artery between the larger metropolitan areas in the region, conveniently passing along the outskirts of proximate cities shown in figure 1. Radial routes are to extend from the cities to the highway. Plans for ultimate improvements, formulated in 1926 by the Chicago Regional Planning Association and the highway engineers of Illinois, Indiana, and Wisconsin, designate a motorway of the parkway type built on a 200-foot right-of-way, and having at least four lanes, as indicated in figure 2. By September 1939, the land for 72 miles of this 161-mile route had been obtained. Negotiations are under way for right-of-way on additional mileage.

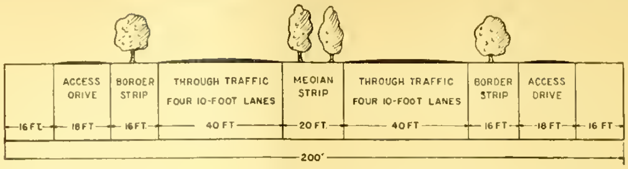

Figure 2.—Cross Section of the Three-State Highway as Originally Planned in 1925. Data Supplied by Chicago Regional Planning Association. Two of the Four Strips of Pavement Ultimately Are To Be 18 Feet Wide, the Other Two 40 Feet Wide. The Latter Would Constitute the Center Strips, Divided by a 20-Foot Median Strip, Each Center Strip Consisting of Four Lanes of Pavement and Serving Through Traffic. On the Outer Side of These Through-Traffic Lanes, a 16-Foot Border Strip Would Be Developed to Separate Them from 18-Foot Access Drives; Along the Outer Edge of These Drives, 16 Feet Would Be Provided for Shoulders and for Future Development. This Would Give Adjacent Property Owners the Right of Access Without Interfering With Through Traffic, Since the Drives Would Be Connected With the Through Lanes at Half-Mile Intervals. It Was Planned That the Through-Traffic Lanes Would Be Paid for Out of General State or County Funds, While the Access Drives Would Be Financed by Special Assessments.

Problems involved in the construction of the Three-State Highway are typical of those which must be solved in any major highway improvement. The section of the highway studied is located in two States, three counties in one State and three in the other. It traverses urban, suburban, and rural territory in which commercial and residential property and agricultural lands are represented.[5] Right-of-way was acquired by donation, purchase, and condemnation. Some marginal or "excess" land was taken, considerably more in Illinois than in Wisconsin. Certain portions of the highway made use of existing roadway. Other portions are relocations.

Local conditions have forced a modification of the original plan of having a uniform 200-foot right-of-way throughout the length of the Three-State Highway. At the time of this investigation, Milwaukee County, Wis., had completed the widening of its portion from an existing 66-foot right-of-way to 160 feet, by acquiring land on either side of the highway to a depth of 47 feet. The right-of-way in Racine and Kenosha Counties, Wis., was limited to 120 feet, but a considerable amount of land was taken for drainage channels and earthwork extending beyond the 120-foot limit. In Lake County, Ill., the highway is built on a 160-foot right-of-way. In Cook and Du Page Counties, Ill., the right-of-way is 200 feet. The plans for ultimate development on each of these widths are shown in figure 3.

Three-State Highway as of September 1939.—The Three-State Highway is a composite of many existing routes, as well as of new sections of roadway, as indicated in figure 4. The highway follows U S 41 south from Milwaukee city limits, through Milwaukee and Racine Counties, Wis., a distance of 25.8 miles to State Highway 43 running east and west across the northern part of Kenosha County, Wis. From this point south to the Wisconsin-Illinois State line, a distance of 8.2 miles, U S 41 was relocated on Bond Issue Route 68 in northern Lake County, Illinois. From the State line, the Three-State Highway proceeds south on U S 41 to and along Bond Issue Route 63, a distance of 13.4 miles. At present, it proceeds southeasterly parallel to and 2½ to 3 miles west of Lake

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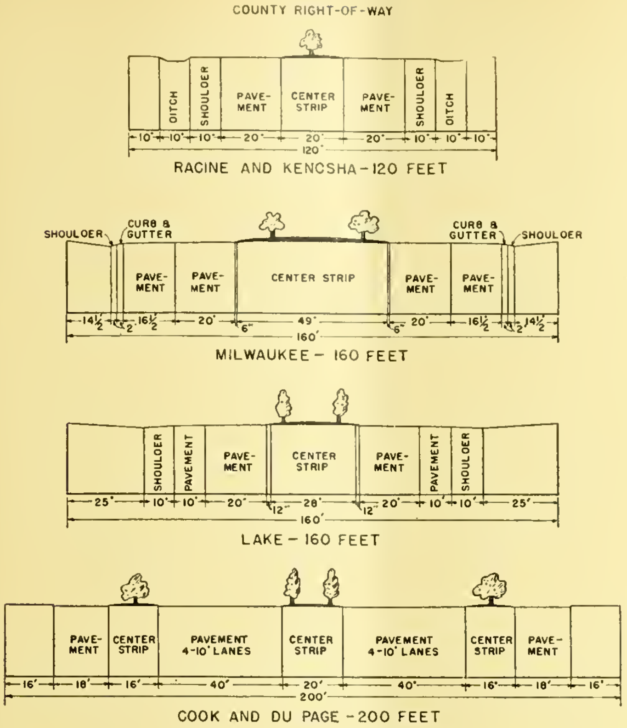
[5] A large portion of the area traversed by the highway is suburban-rural fringe area. East, north, and south of the route lie highly developed urban regions in which are located the cities of Milwaukee, Racine, and Kenosha in Wisconsin and Chicago in Illinois. In such fringe areas, new land uses tend to encroach upon former uses, with the result that agricultural, commercial, residential, semipublic and public uses exist side by side.



RACINE AND KENCSHA - 120 FEET

MILWAUKEE - 160 FEET

LAKE - 160 FEET

COOK AND DU PAGE - 200 FEET

FIGURE 3.—PLAN OF ULIMATE DEVELOPMENT FOR VARYING RIGHT-OF-WAY WIDTHS.

Michigan to Niles Center, Ill., and then southeast and east into Chicago's newly developed Outer Drive at Foster Avenue, about 6.5 miles north of the Chicago Loop.

The Three-State Highway, at its intersection with Bond Issue Route 63, begins a belt route which is still incomplete. In September 1939, the alignment from a point one-half mile north of U S 45 in central Lake County, Ill., southward to a point in the southern part of northern Cook County, had not been surveyed. From the end of the unsurveyed section in the town of Elk Grove, the belt route is constructed along the section lines of Elk Grove to the Cook-Du Page County line, a distance of 2 miles. From the county line, the route proceeds along the section lines of the town of Addison; then follows Bond Issue Route 54 from its junction with Bond Issue Route 64 to the southern Cook-Du Page County line, a distance of 20.4 miles. Beginning at the county line, 1 mile of the belt route is constructed on Bond Issue Route 54 through the Sanitary District of Chicago.[6]

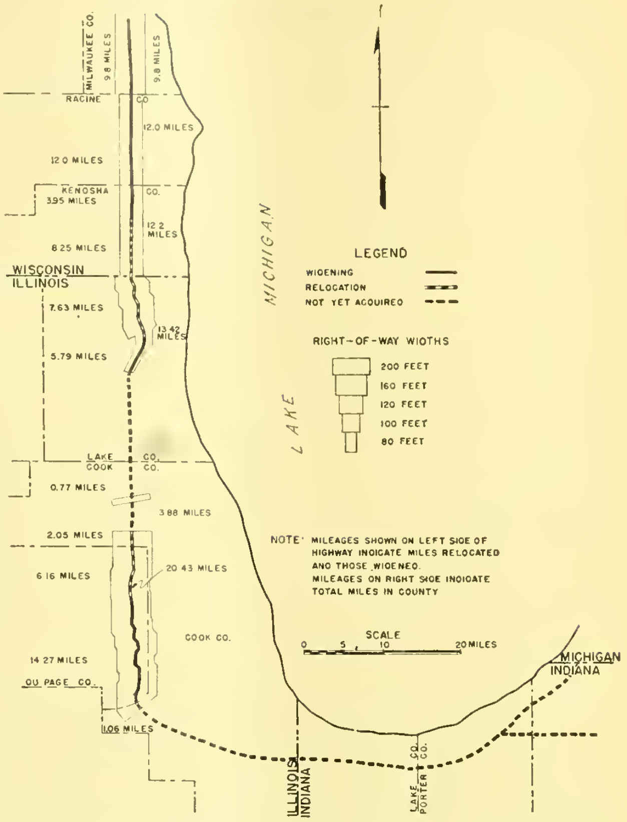


FIGURE 4.—THREE-STATE HIGHWAY RIGHT-OF-WAY ACQUIRED AS OF SEPTEMBER 1939. DATA TAKEN FROM FILES OF HIGHWAY DEPARTMENTS OF THE RESPECTIVE COUNTIES.

TABLE 1.—*Number of parcels acquired*

County	Number of parcels	County	Number of parcels
Grand total	835	Kenosha	51
		Lake	82
Milwaukee	212	Cook	27
Racine	91	Du Page	372

In September 1939, only 72 miles of the originally planned 161-mile route had been completed. The southern terminus was about five-sixths of a mile north of the intersection of State-aid routes 126 and 178 in the northern portion of southern Cook County.

Number of parcels acquired.—By September 1939, right-of-way for 72 miles of the highway had been obtained. This necessitated the acquisition of 835 parcels of real estate, as indicated in table 1, or an average of 11.6 parcels per mile of road. In rural areas, the acquisitions per mile were 4.2 parcels in Kenosha County, 6.1 in Lake County, 7.0 in Cook County, 7.6 in Racine County, and from 4.2 upwards in Milwaukee County.[7] In the urban fringe areas the acquisitions per mile ranged up to 21.6 parcels in Milwaukee County and 18.2 in Du Page County. The properties were largely residential and commercial.

The number of parcels acquired per mile has considerable influence upon acquisition costs, for incidental and condemnation costs vary directly with the number

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[6] In this study information was obtained from the following sources:

I. Wisconsin:

Milwaukee County.—Wisconsin Highway Commission, Madison; Wisconsin Highway Commission, District Office No. 2; Milwaukee County Highway Department; Milwaukee County Regional Planning Department; Milwaukee County Clerk and Register of Deeds; an official appraiser for Milwaukee County; all at Milwaukee.

Racine County.—Wisconsin Highway Commission, Madison; Wisconsin Highway Commission, District Office No. 2, Milwaukee; Racine County Highway Department; Racine County Clerk and Register of Deeds; all at Racine.

Kenosha County.—Wisconsin Highway Commission, Madison; Wisconsin Highway Commission, District Office No. 2, Milwaukee; Kenosha County Highway Department; County Clerk; Register of Deeds; all at Kenosha.

II. Illinois:

Lake County.—Illinois State Department of Public Works and Buildings, Division of Highways, District Office, Elgin; Lake County Highway Department; County Clerk; and Register of Deeds; all at Waukegan.

Cook County.—Illinois State Department of Public Works and Buildings, Division of Highways, District Offices, Elgin and Chicago; Chicago Regional Planning Association, Chicago; Cook County Highway Department; County Clerk; Register of Deeds; all at Chicago.

Du Page County.—Illinois State Department of Public Works and Buildings, District Office, Elgin; State's Attorney's Office; Du Page County Highway Department; Register of Deeds; County Clerk; County Court Clerk; all at Wheaton.

[7] In Racine and Kenosha Counties, more than two-thirds of the land needed for the highway was acquired on one side of the existing highway to avoid excessive damage costs; this practice reduced the number of parcels acquired in these counties.

Table 2.—*Total area of all acquisitions*

County and classification according to population density	Land use Total										
	Total	Agricultural		Commercial		Industrial		Residential		Public and semipublic	
	Area	Area	Percent	Area	Percent	Area	Percent	Area	Percent	Area	Percent
	Acres	*Acres*		*Acres*		*Acres*		*Acres*		*Acres*	
Grand total	1,064.71	718.49	67.48	23.84	2.24	3.63	0.34	224.96	21.13	93.79	8.81
Rural	881.88	657.46	74.55	12.28	1.39	3.63	0.41	129.08	14.64	79.43	9.01
Suburban	92.92	61.03	65.69	11.07	11.91			18.29	19.68	2.53	2.72
Urban	89.91			0.49	0.54			77.59	86.30	11.83	13.16
Milwaukee: Total	110.75	68.32	61.69	5.30	4.79			33.96	30.66	3.17	2.86
Rural	69.87	50.18	71.82	3.23	4.62			15.67	22.43	0.79	1.13
Suburban	40.88	18.14	44.37	2.07	5.06			18.29	44.75	2.38	5.82
Racine: Total	116.49	93.41	80.19	13.05	11.20			3.58	3.07	6.45	5.54
Rural	64.45	50.52	78.39	4.05	6.28			3.58	5.55	6.30	9.78
Suburban	52.04	42.89	82.42	9.00	17.29					0.15	0.29
Kenosha: Rural (also total) [1]	136.11	135.68	99.68	0.42	0.31			0.01	0.01		
Lake: Rural (also total) [1]	166.01	161.89	97.51	0.33	0.20			0.94	0.57	2.85	1.72
Cook: Rural (also total) [1]	78.88	50.56	64.10	2.09	2.65			13.57	17.20	12.66	16.05
Du Page: Total	456.47	208.63	45.70	2.65	0.58	3.63	0.80	172.90	37.88	68.66	15.04
Rural	366.56	208.63	56.92	2.16	0.59	3.63	0.99	95.31	26.00	56.83	15.50
Urban	89.91			0.49	0.54			77.59	86.30	11.83	13.16

[1] No urban or suburban lands acquired.

of acquisitions. Every acquisition, regardless of the size of a tract, requires negotiation, verification of description and title, recording of the conveyances, etc. If condemnation is necessary, legal costs and disbursements, witness fees, appraisal charges, and other expenditures are in proportion to the number of parcels.

Area, land use and population density analysis.—Of the total of 1,065 acres of land acquired, 82.9 percent was rural land, 8.7 percent was suburban territory, and 8.4 percent was within the limits of incorporated villages and cities, as shown in table 2.

The greatest portion of land, both in area and linear frontages, acquired for the highway consisted of agricultural property. Of the total 1,065 acres acquired, 67 percent consisted of agricultural land and 21 percent of residential property. Moreover, of the total highway frontage of 125 miles, 73.4 percent consisted of agricultural and 17.8 percent of residential lands. Commercial, industrial, public and semipublic properties therefore constitute only small percentages of the total. It is significant that commercial property, although it represented but 2 percent of the area acquired, accounted for 20 percent of the entire cost of the acquisitions.

An analysis of population density in terms of land use is of interest. Of the total right-of-way acquired for the Three-State Highway, 83 percent was rural land, 9 percent suburban, and 8 percent urban. Agricultural property is practically confined to rural territory but 61 acres out of the total of 718 acres acquired was in suburban areas. Likewise, land predominantly devoted to residential usage was not restricted to urban and suburban areas. Of a total of 225 acres of land classed as residential, 57 percent was acquired in rural areas, chiefly because of platted property in suburban-fringe districts.

Table 3.—*Area of land acquired by purchase, donation, and by condemnation*

County	Total	Purchase		Donation [1]		Condemnation	
	Area	Area	Percent	Area	Percent	Area	Percent
	Acres	*Acres*		*Acres*		*Acres*	
Grand total	1,064.71	955.39	89.73	44.78	4.21	64.54	6.06
Milwaukee	110.75	99.60	89.94	7.38	6.66	3.77	3.40
Racine	116.49	105.65	90.69			10.84	9.31
Kenosha	136.11	136.11	100.00				
Lake	166.01	154.32	92.96	11.69	7.04		
Cook	78.88	65.92	83.57	10.85	13.76	2.11	2.67
DuPage	456.47	393.79	86.26	14.86	3.26	47.82	10.84

[1] Neither land nor damage costs were involved.

LAND ACQUIRED BY A VARIETY OF METHODS

Table 3 shows the amount of land acquired by donation, purchase, and condemnation.[8] The amount donated is of particular interest in this study because efforts to obtain a large portion of the right-of-way through donation had been made at the outset of negotiations for this improvement. Only 13 percent of the total area involved was acquired by donation. In Milwaukee County 93 percent of the total land acquired was donated, in Lake County 7 percent, in Cook County 14 percent, and in Du Page County 38 percent. No land was donated in Kenosha County or in Racine County. The relatively high percentage of donations in Milwaukee County is accounted for by the county's practice of offsetting the benefits accruing to property as the result of an improvement against the value of the land acquired, one invariably canceling the other. The percentage for Cook County is unreliable because of the comparatively small amount of land involved.

In acquiring more than a thousand acres of land, 6 percent had to be condemned. Milwaukee County made the best showing by acquiring only 3.4 percent through court proceedings. The Cook County sample is inadequate and may be ignored.

Outstanding differences characterize the methods for acquiring rights-of-way in Illinois and Wisconsin. Wisconsin utilizes the so-called statutory award system under which the State or a county awards an amount that is deemed to be the value of land and compensation for damages and tenders it to the property owner. Construction may commence immediately afterwards. If a tender is not accepted, the award is deposited with the county clerk or other designated officer to be held by him in trust for the property owner until accepted or until the procedure is terminated by a formal settlement consummated through other proceedings. In Illinois the State constitution prohibits the taking of private property for public use without just compensation. Such compensation must be ascertained and paid the owner before a property is taken or construction started. The Wisconsin statutory award system expedites the construction of highways and may well serve as a model procedure in the acquisition of lands for public purposes.

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[8] Table 3 is based upon the concept of a donation as involving neither land nor damage cost. In the text discussion, the term "donation" means the conveyance of "bare" land without compensation, even though damages to improvements may be involved.

The constitution of Wisconsin [9] also permits highway authorities to condemn marginal or "excess" lands. Illinois lacks such an organic legal sanction. It is an interesting side light that local units in Illinois acquired much more marginal lands (through voluntary purchase) than did Wisconsin counties. (See p. 264).

ACQUISITION COSTS REPORTED

Land, once the cheapest thing in all the world, is now worth millions of dollars an acre in some areas. Since highway facilities are most urgently needed in the more densely populated sections of our country, where land values are relatively high, it becomes important to know what the rights-of-way for motorways are likely to cost. While sweeping generalizations about land acquisition costs should be avoided, a detailed analysis of the expenditures for the lands acquired for the Three-State Highway is enlightening, and may be indicative of probable costs under similar conditions in other sections.

Three elements constitute the total cost of property acquired by a public authority for a public purpose. These are, the cost of the bare land, the payments for damages resulting from acquisition, and the various incidental fees paid to others than the property owners. Table 4 summarizes these elements of cost on a per-mile basis. Figure 5 presents the data in graphic form.

Various units of measure, the acre, for example, which is a unit of area, and the mile, a unit of length, may be used effectively in presenting comparable data on highway land acquisition. Two linear units have been utilized in this study, the mile and the frontage-mile. The mile represents 5,280 feet of the length of the highway including cross roads and streets. The linear feet of properties fronting on the highway measured along the highway, excluding the frontage of cross roads and streets, is the basis for the frontage-mile. The two sides of the road are considered separately, 1 mile of highway (without cross roads and streets) being equivalent to 2 frontage-miles. Some 72 miles of roadway were acquired for the Three-State Highway, representing the acquisition of 125 miles of frontage.

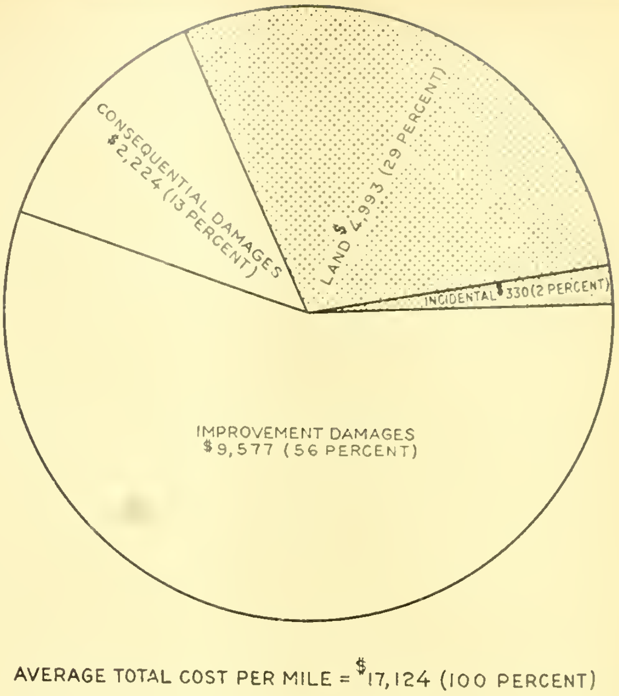

FIGURE 5.—AVERAGE COST PER MILE OF LAND, DAMAGE, AND INCIDENTAL ITEMS.

TABLE 4.—*Summary of costs of land, damages, and incidental fees per mile of highway*

County	Miles of highway	Cost per mile			
		Total	Land	Damage	Incidental
Grand total	71.7	$17,124	$4,993	$11,801	$330
Milwaukee	9.8	41,025	1,015	39,053	957
Racine	12.0	16,185	2,464	13,287	434
Kenosha	12.2	4,426	1,124	3,215	87
Lake	13.4	4,079	1,220	2,810	49
Cook	3.9	18,080	4,680	12,941	459
Du Page	20.4	22,180	13,235	8,671	274

The average total acquisition cost per mile for the entire 72 miles was $17,124. Of this total cost, 29 percent represents the cost of bare land, 69 percent the amount of damages, and 2 percent out-of-pocket expenditures for services incidental to the acquisition. The average total cost ranged from $4,079 per mile in Lake County, Ill., to $41,025 per mile in Milwaukee County, Wis. The comparatively high cost of the Milwaukee County acquisition is due to the rather intensive development that fronted the existing roadway in that county, affirmed by the fact that the damage item alone—$39,053—was almost three times the cost of damage per mile in any of the other counties. The extremely small land cost in Milwaukee County, averaging $1,015 per mile, is the result of a practice of offsetting the potential benefits resulting from a highway improvement against the value of the land taken. Payments for land as such were made to public organizations when the benefits were expected to be of negligible value and to owners of lands condemned when claims of benefits to be offset were apparently not sustained. Table 5 is an analysis of the total cost per acre of all acquisitions according to land use and population density. Figure 6 illustrates right-of-way land and damage costs per mile. Study of this figure will reveal much about Three-State Highway acquisition costs.

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[9] Wisconsin Constitution, Article XI, Section 3a, provides that "the State or any of its cities may acquire by gift, purchase or condemnation lands for establishing, laying out, widening, enlarging, extending, and maintaining memorial grounds, streets, squares, parkways, boulevards, parks, playgrounds, sites for public buildings, and reservations in and about and along and leading to any or all of the same; and after the establishment, layout, and completion of such improvements, may convey any such real estate thus acquired and not necessary for such improvements, with reservations concerning the future use and occupation of such real estate, so as to protect such public works and improvements and their environs, and to preserve the view, appearance, light, air and usefulness of such public works."

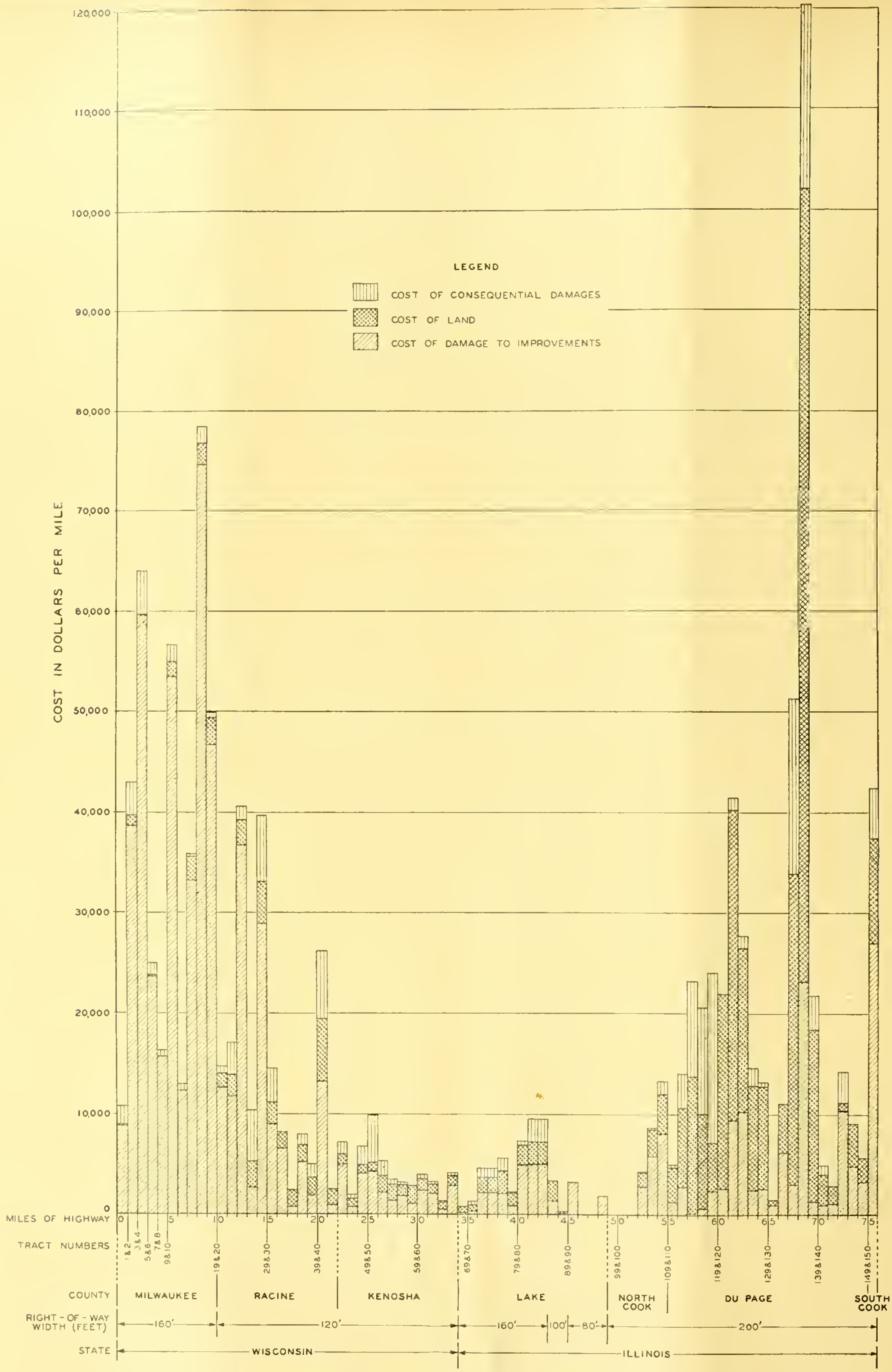

Figure 6.—Land and Damage Costs Per Mile.

TABLE 5.—*Total cost per acre of all acquisition.*

County and classification according to population density	Total		Agricultural		Commercial		Industrial		Residential		Public and semi-public	
	Area	Cost per acre	Area	Cost per acre	Area	Cost per acre	Area	Cost per acre	Area	Cost per acre	Area	Cost per acre
	Acres		*Acres*		*Acres*		*Acres*		*Acres*		*Acres*	
Grand total	1,064.71	$1,154	718.49	$677	23.84	$10,145	3.63	$875	224.96	$1,787	93.79	$1,007
Rural	881.88	901	657.46	633	12.28	14,536	3.63	875	129.08	1,081	79.43	723
Suburban	92.92	2,112	61.03	1,156	11.07	5,555			18.29	2,425	2.53	7,838
Urban	89.91	2,638			0.49	3,787			77.59	2,812	11.83	1,452
Milwaukee: Total	110.75	3,630	68.32	2,745	5.30	21,807			33.96	1,705	3.17	12,945
Rural	69.87	3,703	50.18	2,691	3.23	27,538			15.67	865	0.79	26,842
Suburban	40.88	3,506	18.14	2,895	2.07	12,863			18.29	2,425	2.38	8,332
Racine: Total	116.49	1,667	93.41	893	13.05	6,732			3.58	3,599	6.45	1,563
Rural	64.45	2,193	50.52	1,294	4.05	13,083			3.58	3,599	6.30	1,600
Suburban	52.04	1,016	42.89	420	9.00	3,874					0.15	
Kenosha: Rural (also total) [1]	136.11	397	135.68	358	0.42	12,531			0.01	12,975		
Lake: Rural (also total) [1]	166.01	330	161.89	330	0.35	758			0.94	691	2.85	160
Cook: Rural (also total) [1]	78.88	889	50.56	690	2.09	5,653			13.57	1,557	12.66	183
Du Page: Total	456.47	993	208.63	378	2.65	7,960	3.63	875	172.90	1,789	68.66	591
Rural	366.56	589	208.63	378	2.16	8,907	3.63	875	95.31	957	56.83	412
Urban	89.91	2,638			0.49	3,787			77.59	2,812	11.83	1,452

[1] No urban or suburban lands acquired.

TABLE 6.—*Total cost per mile of frontage of all acquisitions*

County and classification according to population density	Total		Agricultural		Commercial		Industrial		Residential		Public and semi-public	
	Frontage	Cost per mile	Frontage	Cost per mile	Frontage	Cost per mile	Frontage	Cost per mile	Frontage	Cost per mile	Frontage	Cost per mile
	Miles		*Miles*		*Miles*		*Miles*		*Miles*		*Miles*	
Grand total	125.18	$9.812	91.87	$5,298	2.73	$88,590	0.30	$10,585	22.27	$18.054	8.01	$11,792
Rural	104.86	7,580	83.28	4,998	1.66	107,530	0.30	10,585	12.42	11,233	7.20	7,979
Suburban	13.29	14,763	8.59	8,210	0.99	62,116			3.30	13,439	0.41	48,364
Urban	7.03	33,744			0.08	23,198			6.55	33,312	0.40	42,938
Milwaukee: Total	19.32	20,810	11.85	15,826	0.89	129,859			6.05	9,571	0.53	77.423
Rural	11.94	21,669	8.52	15,847	0.53	167,826			2.75	4,929	0.14	151,464
Suburban	7.38	19,420	3.33	15,771	0.36	73,964			3.30	13,439	0.39	50,844
Racine: Total	16.13	12,041	13.53	6,164	1.35	65,077			0.50	25,770	0.75	13,442
Rural	10.22	13,830	8.27	7,907	0.72	73,591			0.50	25,770	0.73	13,810
Suburban	5.91	8,947	5.26	3,424	0.63	55,346					0.02	
Kenosha: Rural (also total)[1]	16.94	3,188	16.89	2,878	0.05	105,261						
Lake: Rural (also total)[1]	27.08	2,021	26.15	2,042	0.05	5,000			0.14	4,613	0.74	616
Cook: Rural (also total)[1]	7.26	9,662	4.96	7,034	0.15	78,764			1.11	19,932	1.04	2,232
Du Page: Total	38.45	11,785	18.49	4,269	0.24	87,896	0.30	10,585	14.47	21,380	4.95	8,194
Rural	31.42	6,872	18.49	4,269	0.16	120,245	0.30	10,585	7.92	11,512	1.55	5,110
Urban	7.03	33,744			0.08	23,198			6.55	33,312	0.40	42,938

[1] No urban or suburban lands acquired.

COST PER MILE VARIED WITH LAND USE AND POPULATION DENSITY

The investigation affirms the principle that over-all cost of property does not vary directly with quantity of land acquired. This is largely because of variations in land use and population density. Although only 8.7 percent of the area acquired was in suburban territory, 16.0 percent of the total cost of $1,228,281 was incurred for suburban land; 8.4 percent was in urban areas and was responsible for 19.3 percent of the cost; and 82.9 percent was in rural sections and accounted for 64.7 percent of the total cost.

Because a mile of the highway may have required lands devoted to various uses and may include portions where no lands at all were taken, a more accurate measure of acquisition costs is the cost per frontage-mile, as indicated in table 6. Although the average cost per frontage-mile for all property acquired was $9,812, the average cost in rural areas was $7,580 per frontage-mile, in suburban areas almost twice as much, and in urban places more than four times the rural average.

The cost varied to an even greater extent with the nature of land use. Costs ranged from $616 per frontage-mile for semipublic property in rural areas in Lake County to $167,826 for a half-mile of commercial property in a rural area of Milwaukee County. The average cost per frontage-mile for all agricultural property was $5,298. Although agricultural property in rural areas was obtained for $4,998 per frontage-mile, agricultural lands acquired in suburban areas were almost double that rate.

The same tendency applies to acquisitions of residential property of which the average cost per frontage-mile was $18,054. Notwithstanding that the cost was $11,233 per frontage-mile for residential lands in rural areas, it amounted to approximately three times as much in urban areas. Residential property was twice as expensive per frontage-mile as agricultural property, and even though the Three-State Highway traverses the undeveloped sections of cities and villages, the cost of residential property in urban localities was approximately three times the cost of residential property in rural areas.

Right-of-way was not acquired through commercial, industrial, public, and semipublic properties in sufficient quantities to justify conclusions concerning cost relationships. The acquisitions analyzed, however, do indicate that cost of lands devoted to such uses may have a decided influence upon total costs. Although but 2.7 frontage-miles of commercial property were acquired, the cost per frontage-mile was $88,590, or more than four times the cost of an equal amount of residential frontage, and more than 17 times the cost of agricultural lands.

These frontage-mile sums when translated into ordinary mileage costs indicate that the cost of acquiring the property necessary for one mile of the highway through rural areas was approximately $15,000, in suburban territory $29,000, and in urban places $67,000. Furthermore, the data indicate that 1 mile of right-of-way through agricultural property averaged approximately $10,500 on the whole route but that its cost in rural areas was about $10,000, and in suburban regions approximately $16,000. The cost per mile of residential property was $22,000 in rural areas, $26,000 in suburban places, and approximately $66,000 in urban centers. Acquisitions of commercial, industrial, public, and semipublic lands were highly dispersed and not in sufficient amounts to warrant extensive analysis.

COSTS OF "BARE" LAND ANALYZED

Expenditures for "bare" land amounted to $358,140 or 29 percent of the total expended for rights-of-way. The land cost was lowest in Milwaukee County—but 2 percent of total expenditures in that county—because of the practice, previously mentioned, of offsetting benefits accruing to property as the result of an improvement against the value of the lands taken. On the other hand, 60 percent of the total was cost of land in Du Page County, Ill., due in large measure to the great amount of platted but unimproved property in that region.

Purchase of 718 acres of agricultural "bare" land entailed an expenditure of $90,763, or a per-acre cost of $126, as shown in table 7. Average cost ranged from $94 per acre in Lake County to $213 per acre in Cook County. The general average cost is somewhat influenced by the Milwaukee County land policy. If acquirements in Milwaukee County are omitted from the aggregate, the land cost averages $137 per acre for relatively high-grade farm lands in "rurban" territory. Excluding Milwaukee County, residential lands averaged $1,107 per acre, the highest cost being $1,871 per acre in urban places in Du Page County, Illinois. Rural lands devoted to all uses, excluding Milwaukee County, averaged $233 per acre, while suburban lands averaged $281 per acre, and urban property $1,708 per acre.

The range in rural land costs per frontage-mile, excluding Milwaukee County, was from $605 in Lake County to $3,713 in Du Page County, as shown in table 8. In residential areas, the cost per frontage-mile, omitting Milwaukee County, ranged from $3,929 in Lake County to $22,162 in Du Page County. The least expensive bare lands in urban and suburban areas cost approximately twice as much as the most expensive land in rural territory. However, since the depth factor has been ignored entirely in this study, all frontage-mile measures of value are of limited significance.

An analysis of unit costs of bare land acquired by purchase, excluding all donations and condemnations, is summarized in table 9. Only lands devoted to agricultural and residential uses were considered because lands put to other uses were acquired so sparingly as to make the resulting sample unreliable. The analysis shows that the average land cost was 3 cents per square yard or $145 per acre for all farm lands; 16 cents per square yard or $774 per acre for residential lands in rural areas; and 42 cents per square yard or $2,033 per acre for residential lands in urban areas.

TABLE 7.—*Cost of bare land per acre*

County and classification according to population density	Total		Agricultural		Commercial		Industrial		Residential		Public and semipublic	
	Area	Cost per acre	Area	Cost per acre	Area	Cost per acre	Area	Cost per acre	Area	Cost per acre	Area	Cost per acre
	Acres		*Acres*		*Acres*		*Acres*		*Acres*		*Acres*	
Grand total	$1,064.71	$336	718.49	$126	23.84	$1,012	3.63	$569	224.96	$943	93.79	$310
Rural	881.88	214	657.46	125	12.28	1,367	3.63	569	129.08	518	79.43	259
Suburban	92.92	170	61.03	139	11.07	571			18.29		2.53	395
Urban	89.91	1,710			0.49	2,084			77.59	1,870	11.83	637
Milwaukee: Total [1]	110.75	90	68.32	20	5.30	645			33.96	19	3.17	1,419
Rural	69.87	126	50.18	25	3.23	1,057			15.67	40	0.79	4,429
Suburban	40.88	28	18.14	8	2.07	2			18.29		2.38	420
Racine: Total	116.49	254	93.41	197	13.05	594			3.58	634	6.45	180
Rural	64.45	231	50.52	199	4.05	355			3.58	634	6.30	184
Suburban	52.04	281	42.89	194	9.00	702					0.15	
Kenosha: Rural (also total) [2]	136.11	101	135.68	101	0.42	100			0.01	200		
Lake: Rural (also total) [2]	166.01	99	161.89	94	0.33	758			0.94	585	2.85	116
Cook: Rural (also total) [2]	78.88	230	50.56	213	2.09	801			13.57	335	12.66	95
Du Page: Total	456.47	592	208.63	150	2.65	4,145	3.63	569	172.90	1,180	68.66	319
Rural	366.56	318	208.63	150	2.16	4,612	3.63	569	95.31	618	56.83	253
Urban	89.91	1,710			0 49	2,084			77.59	1,871	11.83	637

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[1] See discussion and footnote 8 on page 256, concerning donations of bare land.

[2] No urban or suburban lands acquired.

TABLE 8.—*Cost of bare land per mile of frontage [illegible] acquisition*

County and classification according to population density	Total		Agricultural		Commercial		Industrial		Residential		[illegible]	
	Frontage	Cost per mile	Frontage	Cost per mile	Frontage	Cost per mile	Frontage	Cost per mile	Frontage	Cost per mile	Frontage	Cost per mile
	Miles		*Miles*		*Miles*		*Miles*		*Miles*		*Miles*	
Grand total	125.18	$2,861	91.87	$988	2.73	$8,837	0.30	$6,886	22.27	$9,52[illegible]	8.01	[illegible]
Rural	104.86	1,799	83.28	988	1.66	10,109	0.30	6,886	12.42	[illegible]	7.20	[illegible]
Suburban	13.29	1,189	8.59	987	0.99	6,386			3.30		0.41	[illegible]
Urban	7.03	21,865			0.08	12,762			6.55	22,162	0.40	18,8[illegible]4
Milwaukee: Total [1]	19.32	515	11.85	118	0.89	3,842			6.05	[illegible]	0.[illegible]	8,48[illegible]
Rural	11.94	737	8.52	147	0.53	6,443			2.75	230	0.14	21,9[illegible]
Suburban	7.38	156	3.33	41	0.36	12			3.30		0.[illegible]	2,5[illegible]4
Racine: Total	16.13	1,833	13.53	1,359	1.35	5,744			0.50	4,[illegible]34	0.75	1,[illegible]
Rural	10.22	1,460	8.27	1,216	0.72	1,996			0.50	4,534	0.73	1,[illegible]
Suburban	5.91	2,478	5.26	1,583	0.63	10,029					0.02	
Kenosha: Rural (also total) [2]	16.91	809	16.89	809	0.05	840						
Lake: Rural (also total) [2]	27.08	605	26.15	583	0.05	5,000			0.14	3,929	0.74	440
Cook: Rural (also total) [2]	7.26	2,501	4.96	2,166	0.15	11,[illegible]62			1.11	4,[illegible]91	1.04	1,1[illegible]2
Du Page: Total	38.45	7,032	18.49	1,694	0.24	45,767	0.30	6,886	14.47	11,104	4.95	4,436
Rural	31.42	3,713	18.49	1,694	0.16	62,269	0.30	6,886	7.92	7,440	4.55	3,1[illegible]
Urban	7.03	21,865			0.08	12,762			6.55	22,162	0.40	18,824

[1] See discussion and footnote 8 on page 256, concerning donations of bare land.
[2] No urban or suburban land acquired.

DAMAGES FOUND TO BE LARGE

Unlike land costs, damages paid for injury to improvements and betterments because of right-of-way acquisition are not susceptible of unit measurement, and each individual parcel acquisition is a separate problem. Figure 6 indicates the wide variation in damage costs per mile on the Three-State Highway. More than two-thirds of the total amount, $1,228,281, expended for rights-of-way was incurred for damage payments. The average damage cost was $11,801 per mile (table 4).

Damages for legal injuries may be divided into various categories. The principal classifications in highway land acquisition are improvement or betterment damages and consequential damages. Improvement or betterment damages are amounts of money paid for direct damage to structures, trees, fences, lawns, crops, etc., resulting from the acquisition of property. Consequential damages are amounts of money paid a property owner for injury to the remainder of his land as a result of decrease in site value, deprivation of lateral support, grade change, loss of access, severance, etc., caused by a highway development. There is a third category of damages for which no compensation need be made because, in general, the law does not recognize the validity of damage claims for noise, inconvenience, inesthetic arrangements, and the like. This type of damage is called dammum absque injuria.

Table 10 shows the various elements of the two classes of injuries for which payments were made. Damages to structures constituted the largest single element, aggregating almost half a million dollars, the range being from $10,619 to $74,569 per mile (figure 6). Milwaukee County accounted for over half of the damages to structures because of the "shoestring" nature of the existing highway development in that county. It is alleged that these damages would have been considerably greater but for the existence in Milwaukee County of the Highway Widening Ordinance which in 1926 established an extended highway width.[10] The only other large item was damage to trees. This amounted to almost $100,000 on the 72 miles of highway. The highest amounts were incurred in Milwaukee County where tree damage was approximately $7,000 per mile of highway. It should be noted that acquisitions in Racine County and Kenosha County, Wis., were largely confined to the east side of the existing highway in order to avoid large damage payments. Damage costs per frontage-mile are shown in table 11.

TABLE 9.—*Frequency distribution of unit land costs in agricultural and residential areas, excluding donations and condemnations*

Land cost per square yard	Agricultural land		Land cost per square yard	Residential land			
				Rural		Urban	
Cents	*Square yards*	*Percent*	*Cents*	*Square yards*	*Percent*	*Square yards*	*Percent*
0–0.9			0–4.9	207,445	49.24	13,981	4.02
1–1.9	21,524	0.75	5–9.9	45,807	10.87	26,812	7.72
2–2.9	1,468,234	51.28	10–19.9	69,632	16.52	118,388	34.06
3–3.9	162,807	5.69	20–29.9	37,625	8.93	38,599	11.11
4–4.9	982,553	34.32	30–39.9	14,214	3.37	19,490	5.61
5–5.9	181,031	6.32	40–49.9	17,997	4.27	22,406	6.45
6–6.9	222	0.01	50–59.9	12,110	2.87	9,408	2.71
7–7.9	39,073	1.36	60–69.9	3,317	0.79	43,384	12.48
8–8.9	774	0.03	70–79.9	769	0.18	4,134	1.19
9–9.9			80–89.9	234	0.06	9,639	2.77
10 and over	6,959	0.24	90–99.9	1,802	0.43	7,508	2.16
			100–109.9	1,736	0.41	16,080	4.63
			110–119.9	5,198	1.23	5,112	1.47
			120–129.9	265	0.06	888	0.26
			130–139.9	3,070	0.73	57	0.02
			140–149.9			927	0.27
			150–159.9			189	0.05
			160–169.9			881	0.25
			170–179.9			1,207	0.35
			180–189.9				
			190–199.9				
			200–209.9			888	0.26
			210 and over	149	0.04	7,513	2.16
Total	2,863,180	100.00	Total	421,400	100.00	347,521	100.00
Average cost per square yard, cents	2.92		Average cost per square yard, cents	15.58		41.98	

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[10] See Ray E. Behrens, "Highway Widening Procedure Saves Millions," The American City, December 1938, volume 53, No. 12

TABLE 10.—*Analysis of improvement and consequential damages for all acquisitions*

County	Grand total	Improvement damages					Consequential damages			
		Total	Structures	Fences	Trees	Other [1]	Total	Severance	Grade change	Other [2]
Grand total	$846, 470	$686, 963	$475, 030	$27, 323	$97, 676	$86, 934	$159, 507	$70, 543	$44, 933	$44, 031
Total Wisconsin	581, 400	525, 225	359, 687	15, 646	88, 069	61, 823	56, 175	15, 582	11, 229	29, 364
Milwaukee	382, 723	367, 189	255, 946	5, 972	68, 799	36, 472	15, 534	6, 393	2, 568	6, 573
Racine	159, 448	130, 077	92, 195	4, 092	11, 547	22, 243	29, 371	4, 736	7, 879	16, 756
Kenosha	39, 229	27, 959	11, 546	5, 582	7, 723	3, 108	11, 270	4, 453	782	6, 035
Total Illinois	265, 070	161, 738	115, 343	11, 677	9, 607	25, 111	103, 332	54, 961	33, 704	14, 667
Lake	37, 711	28, 960	9, 824	2, 426	2, 010	14, 700	8, 751	8, 316		435
Cook	50, 208	43, 664	39, 445	406	2, 224	1, 589	6, 544	6, 401		143
Du Page	177, 151	89, 114	66, 074	8, 845	5, 373	8, 822	88, 037	40, 244	33, 704	14, 089

[1] Includes payment for injuries to miscellaneous improvements such as culverts, driveways, grading, lawn, sidewalks, etc.
[2] Includes payments for damage to site value, proximity, etc.

TABLE 11.—*Damage cost per mile of frontage of all acquisitions*

County and classification according to population density	Total		Agricultural		Commercial		Industrial		Residential		Public and semi-public	
	Frontage	Cost per mile	Frontage	Cost per mile	Frontage	Cost per mile	Frontage	Cost per mile	Frontage	Cost per mile	Frontage	Cost per mile
	Miles		*Miles*		*Miles*		*Miles*		*Miles*		*Miles*	
Grand total	125. 18	$6, 672	91. 87	$4, 211	2. 73	$76, 893	0. 30	$3, 667	22. 27	$8, 273	8. 01	$8, 032
Rural	104. 86	5, 592	83. 28	3, 903	1. 66	93, 180	0. 30	3, 667	12. 42	5, 586	7. 20	5, 024
Suburban	13. 29	13, 470	8. 59	7, 196	0. 99	54, 973			3. 30	13, 404	0. 41	45, 245
Urban	7. 03	11, 531			0. 08	10, 210			6. 55	10, 783	0. 40	24, 042
Milwaukee: Total	19. 32	19, 810	11. 85	15, 514	0. 89	119, 262			6. 05	9, 336	0. 53	68, 396
Rural	11. 94	20, 197	8. 52	15, 454	0. 53	150, 070			2. 75	4, 455	0. 14	126, 421
Suburban	7. 38	19, 183	3. 33	15, 670	0. 36	73, 905			3. 30	13, 404	0. 39	47, 566
Racine: Total	16. 13	9, 885	13. 53	4, 588	1. 35	58, 288			0. 50	20, 436	0. 75	11, 287
Rural	10. 22	11, 938	8. 27	6, 342	0. 72	70, 655			0. 50	20, 436	0. 73	11, 596
Suburban	5. 91	6, 336	5. 26	1, 831	0. 63	44, 156					0. 02	
Kenosha: Rural (also total) [1]	16. 94	2, 316	16. 89	2, 014	0. 05	101, 660						
Lake: Rural (also total) [1]	27. 08	1, 393	26. 15	1, 435	0. 05				0. 14	714	0. 74	129
Cook: Rural (also total) [1]	7. 26	6, 916	4. 96	4, 639	0. 15	66, 838			1. 11	14, 571	1. 04	962
Du Page: Total	38. 45	4, 607	18. 49	2, 510	0. 24	41, 578	0. 30	3, 667	14. 47	6, 989	4. 95	3, 743
Rural	31. 42	3, 058	18. 49	2, 510	0. 16	57, 262	0. 30	3, 667	7. 92	3, 852	4. 55	1, 958
Urban	7. 03	11, 531			0. 08	10, 210			6. 55	10, 783	0. 40	24, 042

[1] No urban or suburban land acquired.

Consequential damages on the highway aggregated $159,507. The largest single item of consequential damage resulted from severance of property, which occurred when the highway divided a single owner's tract of land into two separate parts, thereby impairing the economic utility of the whole. In some places a farmhouse was separated from the barn and other farm buildings, thereby making movement between them more difficult. On this highway consequential damages were frequently estimated by a rule-of-thumb method, assuming them to be 50 percent of the value of the bare land acquired. The plottage value of subdivided, potential residential property was frequently diminished by the acquisition of the front 50 feet of lots originally 100 feet deep, as illustrated in figure 7. This figure shows a portion of the highway in the village of Bensonville, Du Page County, Ill.

Still another form of consequential damage resulted from changes in elevation of the roadway surface, impairing the means of ingress and egress, one of the fundamental rights incident to the ownership of property. A typical example is shown in figure 8.

INCIDENTAL COSTS AND ADDITIONAL PAYMENTS REPORTED

In addition to land and damage costs, there are other fees incidental to land acquisition. Among them are amounts paid to others than property owners for special services rendered in connection with a land acquisition. They include sums paid to appraisers, commissioners, registers of deeds, county clerks, title-search companies, and others. Out-of-pocket incidental fees averaged $330 per mile of highway and constituted 2 percent of the aggregate expenditures. Table 12 is an analysis of out-of-pocket costs.

Additional payments are sums paid to grantors of property and others for interests in property needed for the highway. They are in addition to amounts paid at the time of original settlement, and are generally the result of inadequate title searching by public authorities. As shown in table 13, more than $22,000 was paid in this fashion, chiefly in Du Page County, Ill. For example, one parcel in that county was acquired originally for what was supposed to be a very reasonable value, $500. Later the county had to pay an additional $4,000 for the partial release of a mortgage. Three

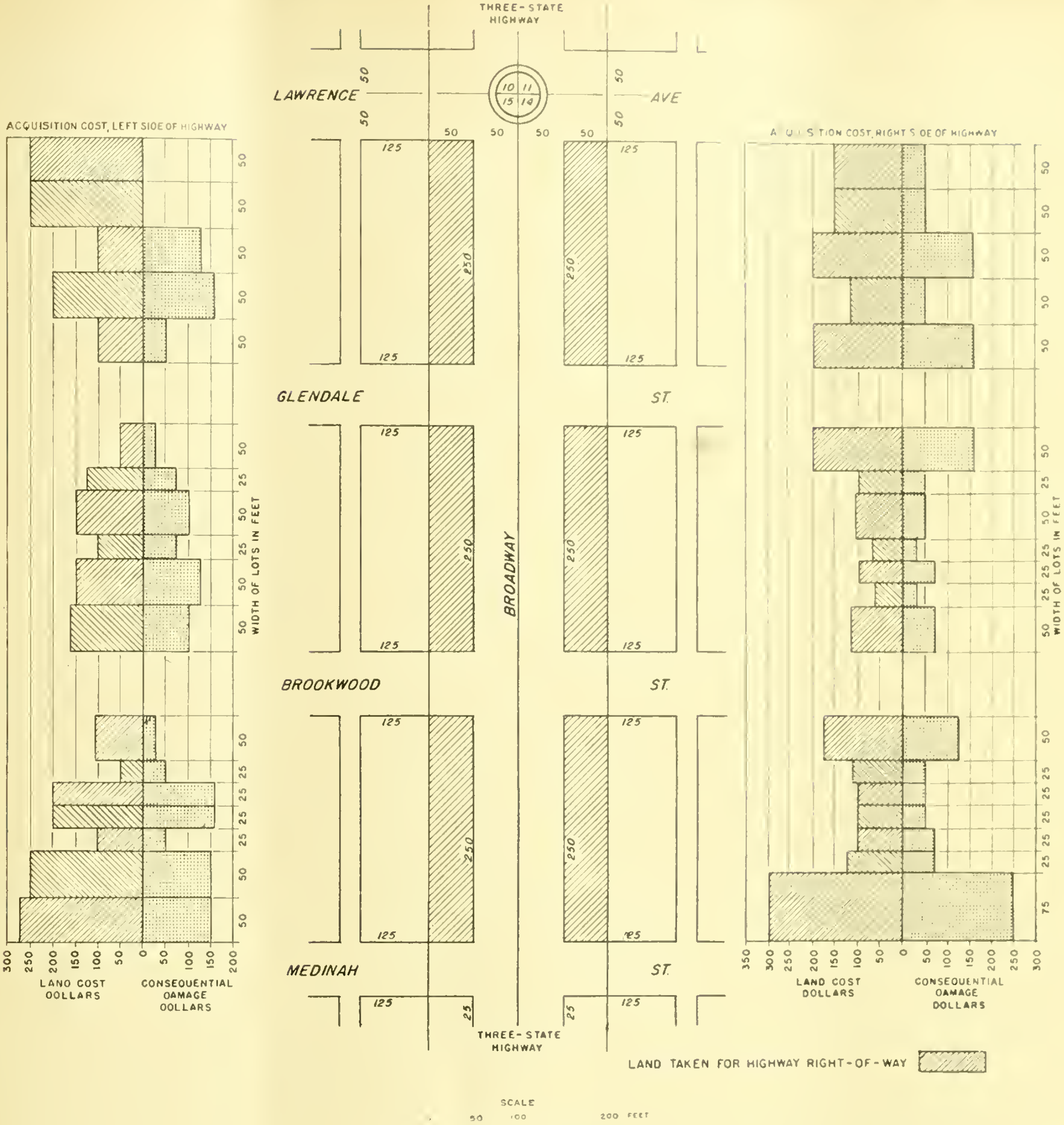

Figure 7.—Extent of Takings from Front of Lots and Costs of Land and Consequential Damages on 1,000 Feet of Highway in the Village of Bensonville, Du Page County, Illinois.

other parcels were acquired at an original total price of $556. After the highway was built, a mortgage holder started foreclosure proceedings and the county had to settle for an additional $4,500. These instances occurred before Du Page County inaugurated its present effective system of title searching. In isolated instances additional payments were made for damages not anticipated. County authorities were willing to give additional compensation as a matter of equity

CONDEMNATION PROCEEDINGS TO BE AVOIDED IF POSSIBLE

If lands cannot be purchased through voluntary negotiation, they must be acquired forcibly through court action. In acquiring more than a thousand acres of land for the Three-State Highway right-of-way, 6 percent had to be condemned. Milwaukee County made the best record by acquiring only 3.4 percent through court proceedings (table 3). Of the 835 parcels acquired, 31 or 3.7 percent had to be condemned. This proportion is not inconsistent with that found on other highway projects. Details concerning costs in condemnation cases are shown in tables 14 and 15.

The condemnation data show that the average condemnation proceedings are likely to add three or four hundred dollars [11] to the total costs of an acquisition. The costs include compensation for witnesses, experts, commissioners, court reporters, and publication services. If there is obstinacy in controversy over values it may be advantageous in some cases for highway authorities to compromise by offering an amount somewhat in excess of the appraised value of a property rather than resort to court action.

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[11] This estimate is probably much lower than [illegible]. In many instances investigated, only approximate data on condemnation [illegible] were [illegible] available

Table 12.—*General and special incidental costs of all acquisitions*

State and County	Total	Cost per frontage mile	General[1]	Special[2]
Grand total	$23, 669	$189	$14, 369	$9, 300
Wisconsin	15, 639	299	7, 651	7, 988
Milwaukee	9, 378	485	4, 213	5, 165
Racine	5, 205	323	2, 531	2, 674
Kenosha	1, 056	62	907	149
Illinois	8, 030	110	6, 718	1, 312
Lake	657	24	657	
Cook	1, 781	245	1, 781	
Du Page	5, 592	145	4, 280	1, 312

[1] Consists of title search, recording fees, and appraisal fees.

[2] Consists of commissioners' fees, witness fees, court costs, and other miscellaneous fees.

Table 13.—*Number and nature of additional payments*

County	Additional payments	
	Number of parcels	Amount
Grand total	[1] 43	$22, 324
For damages	6	4, 225
Additional interest	9	10, 951
Taxes	29	6, 748
Land	2	400
Milwaukee	6	1, 875
For damages	4	1, 725
Additional interest	2	150
Taxes		
Land		
Racine		
Kenosha		
Lake		
Cook		
Du Page	[1] 37	20, 449
For damages	2	2, 500
Additional interest	7	10, 801
Taxes	29	6, 748
Land	2	400

[1] This total is not derived by simple addition since there is overlapping of items.

MARGINAL LAND ACQUISITION USED TO ADVANTAGE

Marginal land acquisition, or "excess condemnation" as it is popularly called, is the acquisition of land and property exceeding the immediate physical needs for a highway improvement, but having definite economic utility in connection with the enterprise. In connection with the Three-State Highway, 42 parcels of marginal land were purchased, aggregating 25.6 acres, at an estimated aggregate cost of $30,823. The land was largely in Du Page County, Ill. Of this land, 14.7 acres were traded for other lands needed for highway purposes, 0.6 acre was sold, 2.7 acres were transferred to other governmental agencies, and 7.6 acres remained in county ownership, as indicated in table 16.

Table 14.—*Cost histories of condemnation cases*

County, tract and parcel number	Amount offered	Amount asked	Final settlement	Incidental cost	Aggregate cost
Milwaukee:					
15.1			$2, 050	$572	$2, 622
16.8	$250		320	295	615
16.9, etc	7, 500	$20, 000	8, 423	1, 695	10, 118
17.10	6, 500	14, 000	7, 940	1, 609	9, 549
17.11	7, 500	15, 000	8, 919	2, 676	11, 595
20.2	3, 500	9, 000	3, 450	1, 453	4, 903
Racine:					
21.6	300	1, 000	484	355	839
21.7	2, 098		2, 750	20	2, 770
23.8, 25.1	1, 500	3, 500	2, 669	517	3, 186
24.6	2, 750		3, 916	226	4, 142
25.6	238	1, 000	1, 000	1	1, 001
25.9, 27.1	4, 214	8, 320	5, 499	469	5, 968
27.2, 27.3	1, 000	5, 000	2, 051	342	2, 393
28.5	500	5, 000	2, 164	322	2, 486
28.6	1, 200	5, 000	2, 257	315	2, 572
30.2	1, 000		2, 500	1	2, 501
Cook, 109.5	317		422	89	511
Du Page:					
114.9			250	111	361
115.7			100	43	143
115.13	200		300	33	333
116.1, 116.2			250	43	293
117.19, etc	20, 000	80, 000	29, 000	547	29, 547
126.3			1, 676	130	1, 806
129.1B			4, 875	30	4, 905
135.4			3, 999	80	4, 079
135.6			3, 665	80	3, 745
137.23			2, 300	20	2, 320
138.22			2, 500	22	2, 522
139.1			4, 956	205	5, 161
146.1			3, 747	30	3, 777
147.6-148.3			2, 826	230	3, 056

Surplus lands were purchased for a variety of reasons. In Milwaukee County, Wis., marginal land was purchased and later traded to obtain a portion of a school district property needed for highway right-of-way. The county had purchased land contiguous to the highway and traded it for a portion of school district property lying within the proposed right-of-way.

In Du Page County, Ill., the highway damaged the sewage disposal plant of the city of Elmhurst. The county obtained a large tract of land near the city limits, retained the portion necessary for highway right-of-way and traded a portion of the balance for the land of the city's sewage disposal plant and for land being held for a potential city park. The county purchased another tract which it expected to trade for other lands needed for right-of-way, but the deal was never consummated and the county still retains this marginal land. Five other tracts were purchased because of the possibility of a future grade separation. A total of 2.7 acres of marginal lands was donated by the county to the Illinois State Forest Preserves. In five instances surplus land was sold to private individuals.

Some surplus land was purchased by Du Page County to avoid the payment of large amounts of consequential

Table 15.—*Area and cost of parcels acquired through condemnation*

County	Total			Elements of cost											
				Land	Damage			Incidental							
	Number of parcels	Area in acres	Cost	Total	Total	Improvement	Consequential	Total	Recording	Appraisal	Title search	Stenographer	Commissioner	Witness	Miscellaneous
Grand total	31	64. 54	$129, 813	$36, 959	$80, 298	$39, 552	$40, 746	$12, 556	$126	$4, 725	$99	$247	$2, 695	$3, 851	$813
Milwaukee	6	3. 77	39, 401	5, 296	25, 806	25, 637	169	8, 299	2	3, 338	7		2, 025	2, 849	78
Racine	10	10. 84	27, 858	2, 158	23, 131	12, 788	10, 343	2, 569	1	983	12	247	670	627	29
Kenosha															
Lake															
Cook	1	2. 11	512	422				90	13	29	48				
Du Page	14	47. 82	62, 042	29, 083	31, 361	1, 127	30, 234	1, 598	110	375	32			375	706

Table 16.—*Analysis of marginal land acquisition*

County, tract and parcel numbers	Aquisition		Disposition
	Area in square yards	Estimated cost	
Milwaukee:			
4.5	780	$360	Held.
12.6	3,652	1,499	Traded.
Racine: 28.3	431	18	Held.
Kenosha	----	----	
Lake: 89.1, 90.1	5,313	110	Held, remnant.
Cook: 151.12, etc	44	2	Held.
Du Page:			
116.19, 116.20	363	148	Held.
116.21	362	130	Held.
121, 122.11c	774	56	Held.
122.23, etc	3	2	Held for future grade separation.
122.24	181	196	Held for future grade separation.
122.25	130	117	Held for future grade separation.
122.26	155	149	Held for future grade separation.
122.27	782	1,018	Held for future grade separation.
123.12	1,310	129	Held.
123.13	66,792	7,529	Traded.
125.3	3,550	50	Held.
125.6	12,889	572	Donated to forest preserve.
136.26	329	331	Held.
136.27	953	330	Held.
136.29	953	300	Traded.
137.9	10	11	Held, remnant.
137.10	69	85	Held, remnant.
137.11	12,541	8,002	Held.
137.12	51	55	Held.
138.7	17	39	Held.
138.8	916	1,930	Held, offer to trade rejected.
138.10	314	330	Held.
138.11	407	665	Sold.
138.13	751	656	Sold.
138.14	731	644	Sold.
138.16	610	696	Sold.
138.17	596	571	Sold.
138.18	580	997	Held.
138.19	1,674	1,365	Held.
139.3	111	31	Held.
139.4	167	46	Held.
139.8	316	91	Held.
139.10	2,778	863	Held.
140.10	250	198	Held.
140.14	534	154	Held.
140.19	24	21	Held.
140.21	929	267	Held.
Grand total	124,121	30,823	

damage claimed because of severance and impairment of plottage values. In several instances where Du Page County felt that the probable cost of "bare" land plus the damages, including those for consequential injuries, approached the value of the entire tract or parcel, the county purchased the whole tract. Figure 9 shows examples of such takings. Because of the limitations of the sample and the many variables involved, it is unwise to draw conclusions from the marginal land activities in Du Page County. All marginal land was acquired through purchase.

HIGHWAY WIDENING VS. HIGHWAY RELOCATION

The attention of highway authorities has been focused for some time upon the economics of highway widening and highway relocation. Analysis of the cost of widening and relocating sections of the Three-State Highway seems to indicate that land acquisition costs of widening are considerably greater than those of relocation. The most striking illustrations of this difference in cost is found in Kenosha County, Wis., and Du Page County, Ill.

Four miles of the highway in northern Kenosha County coincided with the existing U S 41, which was widened from 66 feet to 120 feet. A 5-mile section in that county was on new location with a 120-foot right-of-way. The total cost on the widened portion averaged $6,404 per mile, of which $5,521 constituted damage payments. The total cost on the relocated portion averaged only $3,752 per mile, of which $2,260 constituted damage costs. It was expected that the land costs for the 120-foot width required on new location would exceed those for the 54-foot width required for widening. The areas traversed by these sections of highway were similar in all essential respects, and the properties were acquired at the same time by the same agencies under similar conditions.

Another instance of the relative economy of relocation is found in Du Page County, Ill. Figure 10 shows the location and surrounding conditions. A considerable portion of the existing street, 100 feet wide, in the city of Bensonville, was widened to 200 feet, 50 feet

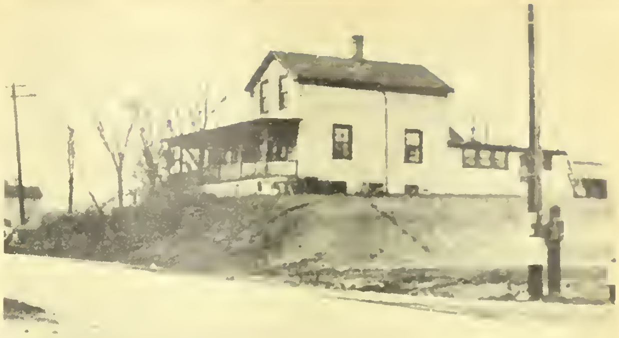

Figure 8.—An Example of Consequential Damage Resulting from a Change of Grade. Thirty-Four Hundred Dollars Was Allowed as Damages to This Corner Property, in the Southern Portion of Milwaukee County.

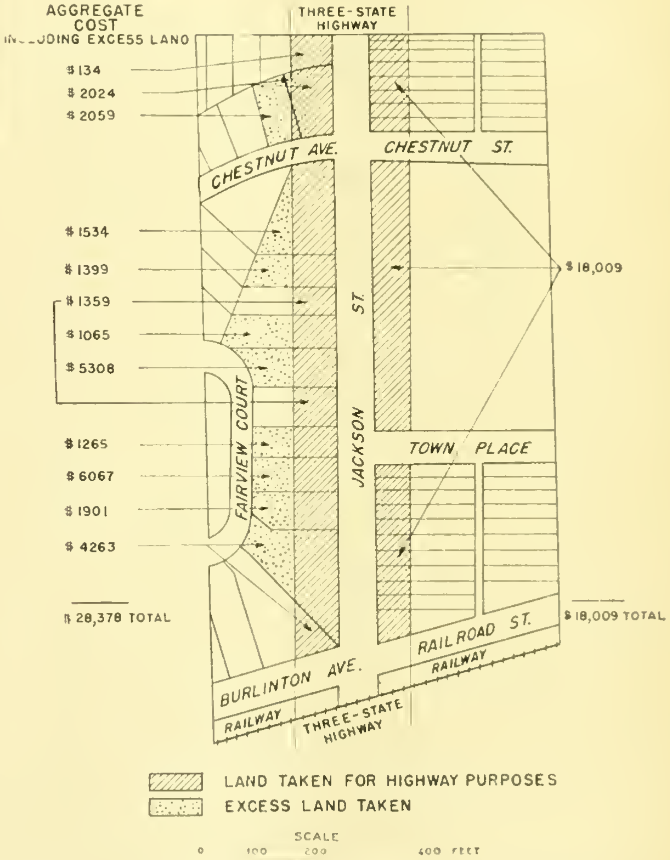

Figure 9.—Location of Excess Land Acquired on 1,000 Feet of Highway in the Village of Clarendon Hills, Du Page County.

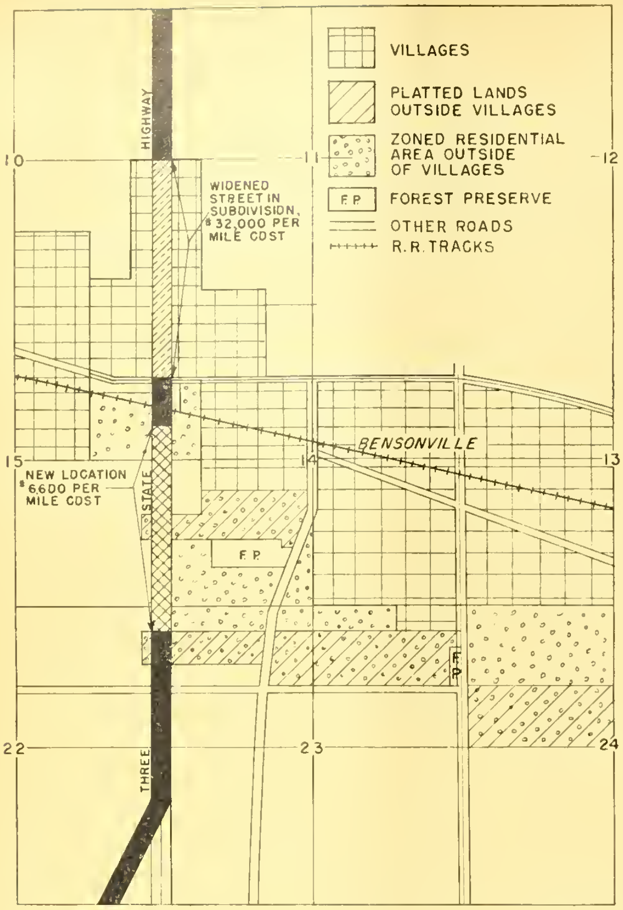
VILLAGES
PLATTED LANDS OUTSIDE VILLAGES
ZONED RESIDENTIAL AREA OUTSIDE OF VILLAGES
F.P FOREST PRESERVE
OTHER ROADS
R.R. TRACKS
HIGHWAY
10
11
12
WIDENED STREET IN SUBDIVISION, $32,000 PER MILE COST
BENSONVILLE
15
14
13
NEW LOCATION $6600 PER MILE COST
STATE
F.P
F.P
22
23
24
THREE

Figure 10.—Comparison of Cost of Sections of the Highway, One on New Location and the Other on an Existing Road That Was Widened.

being taken on each side, at a total cost of approximately $32,000 per mile. This was "raw" subdivision territory, without other development in the area than the startling presence of occasional fire hydrants in vacant lots. Immediately south of the tracks of the Chicago, Milwaukee, St. Paul and Pacific Railroad, the highway was relocated through unplatted territory for a distance of some 3,000 feet. A 200-foot right-of-way was obtained at an approximate total cost of $6,600 per mile. The nature of the land traversed by these two sections of highway was the same in all essential respects except that the property taken for widening was subdivided land and that taken for relocation was unplatted. Acquisition costs for widening, on a per mile basis, were almost five times as great as those for relocation.

GLOSSARY OF TERMS

Terms used in this study are defined as follows:

1. *Access, right of.*—The right of reasonable and convenient ingress to and egress from property to a given highway in addition to the public easement in that highway. This is a right incident to the ownership of abutting land and generally cannot be impaired or taken away without just compensation.

2. *Agricultural land.*—Land and structures used predominantly for farming or truck gardening.

3. *Commercial land.*—Land and structures used predominantly for retail or wholesale business.

4. *Commissioner's fees.*—Amounts of money paid a person acting as a member of a board in adjudicating the compensation to be paid a property owner as a result of land acquisition for a highway enterprise.

5. *Consequential damages.*—Amounts of money paid to a property owner for damages to the remainder of his land as a result of decrease in site value, deprivation of lateral support, grade change, loss of access, severance, etc., caused by a highway development.

6. *Decrease in site value.*—A consequential damage resulting from the acquisition of land in such a way as to prevent or hinder the continuation of a former land use.

7. *Deprivation of lateral support.*—A consequential damage resulting from the construction of grade separations or the excavation of deep cuts which may destroy natural support and cause earth to crumble, sink, or fall away.

8. *Donation.*—The voluntary conveyance of an interest or title in realty without the passing of any real consideration to the grantor.

9. *Grade change.*—A change in the elevation or level of a highway causing it to be above or below the elevation of an existing roadway or of abutting property.

10. *Improvement damages.*—Amounts of money paid for damage to structures, trees, fences, lawns, crops, etc., caused by a given acquisition.

11. *Incidental costs.*—Amounts of money paid to others than property owners for special services rendered in connection with an acquisition. Sums paid to appraisers, commissioners, registers of deeds, county clerks, title search companies, and others, are included.

12. *Industrial land.*—Land and structures used predominantly in the production of goods with labor and capital.

13. *Marginal land acquisition.*—Acquisition of land and property exceeding the immediate physical needs for a highway improvement but having definite economic utility in connection with such improvement. Popularly called "excess condemnation."

14. *Plottage value.*—That element of the worth of a tract of land which exists because of its suitability in size and quality for a given use.

15. *Population density.*—The number of persons per square mile of territory.

16. *Residential land.*—Plotted land and structures predominantly used for present or potential living purposes.

17. *Rural areas.*—Unincorporated areas having a population density of fewer than 100 persons per square mile.

18. *Rurban areas.*—Generally, rural lands enjoying urban services; the term is an amalgam of "rural" and "urban," and represents the infusion of urban culture and interest into rural life.

19. *Semipublic and public lands.*—Land and structures devoted to restricted or nonrestricted common or general use or enjoyment.

20. *Severance damage.*—A form of consequential damage resulting from the division of a tract of land by a highway enterprise in such a manner as to affect adversely the normal use of the residual areas.

21. *Structure damage.*—Amounts of money paid for the moving or destruction of any building or structure or portion thereof in connection with a highway enterprise.

22. *Suburban areas.*—Unincorporated areas having a population density of more than 100 persons per square mile.

23. *Urban areas.*—All incorporated villages and cities.

BENDING AND DIRECT STRESS IN REINFORCED CONCRETE

By W. P. LINTON, Senior Highway Bridge Engineer, and THOMAS P. REVELISE, Associate Highway Bridge Engineer[1]

Several methods for the design and analysis of rectangular sections of reinforced concrete subject to bending and direct stress are in current use. They require the use of diagrams, nomographic charts, and analytical procedures. Perhaps the most familiar method is that described in Hool and Johnson's "Concrete Engineers' Handbook" based on the use of diagrams. Each method has advantages in solving certain design problems but apparently none is sufficiently general for satisfactory use in solving all of the wide variety of problems presented in every day practice.

Most diagrams are useful mainly in analyzing, or checking, designs, rather than in original design. One must first assume the percentage of reinforcement and subsequently check this assumption. If the assumed percentage is found to be inadequate, or uneconomical, the procedure must be repeated. In checking sections containing reinforcement for tension and compression, it is generally necessary to refer to several diagrams for a complete solution. Most diagrams are further restricted in use to problems involving a few fixed proportions of reinforcement, such as equal amounts of steel in top and bottom of a section, and steel in compression one-half that in tension, etc. Often in actual practice the amount of steel required in tension is not an exact multiple of the amount in compression, and in many cases relatively small variations in the amount of reinforcement produce substantial variations in the stresses.

Heretofore analytical procedures have been much favored by engineers for designing sections with tensile and compressive reinforcement to carry bending and direct stress. They are used to some extent for analysis. The design procedure consists usually in the assumption of f_c, the compressive unit stress in the extreme fiber of concrete, and p', the ratio of the area of compressive reinforcement to the effective area of the section, bd, and computation of p, the ratio of the area of tensile reinforcement to the effective area of the section, and f_s, the tensile unit stress in the longitudinal reinforcement, such that $\Sigma H=0$ and $\Sigma M=0$ for the section. The limitation inherent in this and similar analytical procedures arises from the fact that with the dimensions of the section, the moment, the direct force and f_c fixed, an infinite number of consistent values of p, p', f_s, and f'_s exist. For example, suppose a rectangular section 18 inches wide and 20 inches deep were subjected to a bending moment of 800,000 inch-pounds and a direct force of 50,000 pounds, and that f_c were arbitrarily fixed at 1,000 pounds per square inch. The distance from centerline of steel to face of concrete is 2 inches. The following are three of the unlimited number of combinations that satisfy the given conditions:

(*a*) $p=0.0145$; $p'=0.0000$; $f_s=7,800$ pounds per square inch; $f'_s=0.000$ pounds per square inch.

(*b*) $p=0.0075$; $p'=0.0058$; $f_s=12,700$ pounds per square inch; $f'_s=7,750$ pounds per square inch.

(*c*) $p=0.0040$; $p'=0.0140$; $f_s=21,000$ pounds per square inch; $f'_s=6,900$ pounds per square inch.

Obviously, a rational design by conventional methods requires more than one determination to find the optimum combination, making the procedure tedious and time-consuming.

The diagrams presented as plates 1 to 5 are believed to provide a unique method for the design and analysis of rectangular sections of reinforced concrete subject to the following conditions:

(*a*) Simple bending, with tensile reinforcement only.

(*b*) Simple bending, with tensile and compressive reinforcement.

(*c*) Bending and direct stress, tension on part of the section, with tensile reinforcement only.

(*d*) Bending and direct stress, tension on part of the section, with tensile and compressive reinforcement.

These conditions cover most of the problems encountered in the design of rectangular sections of reinforced concrete, including simple beams, eccentrically loaded columns, rigid frames and arches (in which the usual procedure is to analyze an element of unit width) culverts, and other indeterminate type structures.

The diagrams have received extensive trial in the design of reinforced concrete highway bridges, and have been found to reduce by more than one-half the amount of time usually consumed in similar computations by other methods. It is believed that they will save considerable time for concrete designers and checkers, particularly those whose work occasionally involves making a number of determinations of this type.

Although there are numerous aids in solving problems in simple bending, including tables, diagrams and formulas, problems in this category may be solved so readily by use of the diagrams that illustrative examples will be given. The examples are arranged in the order of the conditions stated above, with an example in the design procedure followed by an example in the analytical procedure. To use the curves in design it is not necessary to go into the derivation of the formulas from which they were developed. The examples given illustrate methods of use. However, many readers will wish to trace the mathematical development of the methods in order to judge their accuracy.

DERIVATION OF GENERAL EQUATIONS

The major point of difference between this and other methods lies simply in the generalization of conventional formulas. The accepted theory of bending in reinforced concrete is adhered to without modification. Standard symbols are used throughout with the exception of B and C which represent composite terms having no exact counterpart in current usage. The standard symbols are defined as follows:

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[1] The basic idea for the diagrams presented in this article was developed by W. P. Linton. Thomas P. Revelise collaborated in further improvements and prepared the diagrams and discussion for publication. Acknowledgment is due Logan L. Ratliff, associate highway bridge engineer, for valuable assistance with the computations and lay-out work.

f_c=compressive unit stress in extreme fiber of concrete, pounds per square inch.

f_s=tensile unit stress in longitudinal reinforcement, pounds per square inch.

f'_s=compressive unit stress in longitudinal reinforcement, pounds per square inch.

p=ratio of area of tensile reinforcement to effective area of section.

p'=ratio of area of compressive reinforcement to effective area of section.

n=ratio of modulus of elasticity of steel to that of concrete.

k=ratio of distance between extreme fiber of concrete in compression and neutral axis, to the effective depth, d.

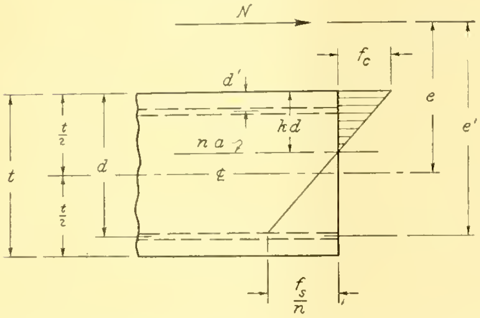


Figure 1.—Diagram for Use in Deriving Equations.

Referring to figure 1, and utilizing two of the conditions of static equilibrium, $\Sigma M=0$ and $\Sigma H=0$, the following may be deduced:

$$\frac{1}{2}f_cbkd^2\left(1-\frac{1}{3}k\right)+nf_cp'bd^2\left(\frac{k-\frac{d'}{d}}{k}\right)\left(1-\frac{d'}{d}\right)=Ne'$$

$$\frac{1}{2}k\left(1-\frac{1}{3}k\right)+np'\left(\frac{k-\frac{d'}{d}}{k}\right)\left(1-\frac{d'}{d}\right)=\frac{Ne'}{f_cbd^2}\text{--------}\quad(1)$$

$$\frac{1}{2}f_cbkd+nf_cp'bd\left(\frac{k-\frac{d'}{d}}{k}\right)-nf_cpbd\left(\frac{1-k}{k}\right)=N$$

$$\frac{1}{2}k+np'\left(\frac{k-\frac{d'}{d}}{k}\right)-np\left(\frac{1-k}{k}\right)=\frac{N}{f_cbd}\text{-------------}\quad(2)$$

Multiplying (2) by $1-\frac{d'}{d}$:

$$\frac{1}{2}k\left(1-\frac{d'}{d}\right)+np'\left(\frac{k-\frac{d'}{d}}{k}\right)\left(1-\frac{d'}{d}\right)$$
$$-np\left(\frac{1-k}{k}\right)\left(1-\frac{d'}{d}\right)=\frac{N}{f_cbd}\left(1-\frac{d'}{d}\right)\text{----}\quad(3)$$

Subtracting (3) from (1):

$$-\frac{1}{6}k^2+\frac{1}{2}k\frac{d'}{d}+np\left(\frac{1-k}{k}\right)\left(1-\frac{d'}{d}\right)$$
$$=\frac{Ne'}{f_cbd^2}-\frac{N}{f_cbd}\left(1-\frac{d'}{d}\right)\text{----}(4)$$

$$\text{from (1): } p'=\frac{\frac{Ne'}{f_cbd^2}-\frac{1}{2}k\left(1-\frac{1}{3}k\right)}{n\left(\frac{k-\frac{d'}{d}}{k}\right)\left(1-\frac{d'}{d}\right)}\text{----------------}\quad(5)$$

$$\text{from (4): } p=\frac{\frac{N}{f_cbd}\left(\frac{e'}{d}-1+\frac{d'}{d}\right)+\frac{1}{2}k\left(\frac{1}{3}k-\frac{d'}{d}\right)}{n\left(\frac{1-k}{k}\right)\left(1-\frac{d'}{d}\right)}\text{------}\quad(6)$$

Let $\frac{Ne'}{f_cbd^2}=B$, and $\frac{N}{f_cbd}\left(\frac{e'}{d}-1+\frac{d'}{d}\right)=C$

$$\text{from (5): } p'=\frac{k}{n\left(k-\frac{d'}{d}\right)\left(1-\frac{d'}{d}\right)}\left[B-\frac{1}{2}k\left(1-\frac{1}{3}k\right)\right]\text{--}\quad(7)$$

$$\text{from (6): } p=\frac{k}{n(1-k)\left(1-\frac{d'}{d}\right)}\left[C+\frac{1}{2}k\left(\frac{1}{3}k-\frac{d'}{d}\right)\right]\text{--}\quad(8)$$

Equations (7) and (8) were used in the form given for plotting the curves, and five individual diagrams were prepared, based on the value of $n=10$, and values of $\frac{d'}{d}=0.05$, 0.10, 0.15, 0.20, and 0.25. This fixed the values of n and $\frac{d'}{d}$ for each diagram, leaving as variables p, p', k, B and C. Since the values of p, p', and k are interdependent, it was possible to plot a series of curves for various values of B with values of p' as abscissas and values of k as ordinates, and adjacent to this, a series of curves for C with values of p as abscissas and the k-axis common to both diagrams. This arrangement, with the related stress curves, results in a diagram of unusual versatility and simplicity of operation.

PROCEDURE IN DESIGN AND ANALYSIS

Using the known data in a given problem, assign the desired value of f_c and compute values of B and C using the simplified formulas given on each diagram. Select the diagram based on the proper value of $\frac{d'}{d}$ and place a straightedge (a transparent triangle is suggested) parallel to the horizontal grid lines and within the range of curves representing computed values of B and C. At the intersections of this horizontal line with curves representing computed values of B and C read the values of corresponding abscissas for values of p and p'. Locate the intersection of the horizontal line with the ordinate through the assigned value of f_c at the right of the diagram. The curve passing through this point (actual or visualized as an interpolation) indicates the appropriate value of f_s. The value of f'_s is similarly determined using the ordinate for f_c at the left of the diagram. Shift the straightedge up or down, maintaining it parallel to the horizontal grid lines, and note by inspection the variations in the values, then select the desired group of values. In all uses of a straightedge on the diagrams the straightedge is kept horizontal or parallel to the x-axis.

When tensile reinforcement only is used, a diagram may be selected at random for solution of the problem without regard to the particular value of $\frac{d'}{d}$ on which it is based. This applies to problems in bending and direct stress as well as those involving only simple bending.

The procedure for problems in simple bending is exactly the same as that set forth for problems in bending and direct stress except that in all cases of simple bending, $B=C$, and the expression for B (or C) becomes

$$\frac{M}{f_cbd^2} \text{ instead of } \frac{Ne'}{f_cbd^2}.$$

The analytical procedure requires an approach differing from that used in design, but its application is no less simple and general. One of the factors sought, f_c, is also contained in the expressions for B and C, hence these terms cannot be evaluated as in the design procedure. It is evident, however, that since f_c has one value only in the design to be analyzed, the ratio, $\frac{B}{C}$, likewise can have but one value. This ratio may be evaluated from the simplified formula given on each diagram. For example, suppose a section is to be analyzed for which the value of $\frac{d'}{d}=0.10$. Looking at the top of the diagram based on this value we observe $C=B\left(\frac{e'-0.90d}{e'}\right)$. For bending and direct stress, therefore, the expression for $\frac{B}{C}$ would become simply the inverse of the expression in parentheses, or $\frac{e'}{e'-0.90d}$. Since $B=C$ in all cases of simple bending, the value of $\frac{B}{C}$ for problems in this category becomes unity.

Having evaluated $\frac{B}{C}$, place a straightedge parallel to the horizontal grid lines, shift vertically along the two ordinates through the given values of p and p', and, by trial, locate the position at which simultaneous values of B and C give the computed $\frac{B}{C}$ ratio. The value of B indicated at this position is inserted in the equation $B=\frac{Ne'}{f_cbd^2}$ (for bending and direct stress), or $B=\frac{M}{f_cbd^2}$ (for simple bending), and f_c is computed. Keep the straightedge in the same position and locate its intersections with ordinates through the computed value of f_c at the right and left of the diagram. The curves (actual or visualized) passing through these points indicate values of f_s and f'_s.

In applying this procedure, it will be observed that the values of $\frac{B}{C}$ converge rapidly in a continuous function, and that the desired ratio may be obtained quickly in all cases.

In using the diagrams to design sections requiring tensile reinforcement only, one occasionally finds a case in which the steel, rather than the concrete, controls the design, that is, having assumed f_c and evaluated B, f_s is observed to exceed the allowable value. For that reason, the following procedure is suggested for design involving tensile reinforcement only.

Assign the allowable value for f_c, and compute B, preferably on the slide rule. Manipulate the rule in such manner that the last operation in obtaining the value of B is division by f_c. Before evaluating C, place the straightedge horizontally so it will pass through the point where the curve representing the tentative value of B is intersected by the ordinate for the value of $p'=0$ and check f_s at the intersection of the straightedge with the ordinate through assumed value of f_c. If f_s is found to exceed the allowable value, shift the slide rule setting to a lower value of f_c, place the straightedge to pass through the point indicated by the new value of B and read f_s at the intersection of the straightedge with the ordinate through new value of f_c. In this manner, the diagram position for the desired steel stress is obtained rapidly. Now compute the value of C, and obtain p at the intersection of the straightedge in its final position with the curve corresponding to the computed value of C.

Values of k, which may be read on the center vertical axis, are not essential to the use of the diagram in either design or analysis, but are indicated as a convenience to the user who may require these data occasionally for special purposes.

The use of the diagrams is more clearly demonstrated by actual illustrative examples, several of which follow. A careful reading and comparison of these examples will show the simplicity of use and a general similarity of application to all cases. The most involved problems may be solved in approximately the same amount of time as that required for the simplest

Example 1.—Simple Bending With Tensile Reinforcement Only; Balanced Design.—Given: A member is subjected to a bending moment of 1,000,000 inch-pounds. Design the section with tensile reinforcement only, such that $f_c=800$ pounds per square inch and $f_s=18,000$ pounds per square inch.

Solution: Since tensile reinforcement only is to be used any one of the diagrams may be selected.

Place a straightedge parallel to the horizontal grid lines so it will pass through the intersection of lines representing $f_c=800$ pounds per square inch and $f_s=18,000$ pounds per square inch. On the vertical line at which $p'=0$ (the center axis) read $B=.138$. For simple bending, $B=C$. Therefore follow back along

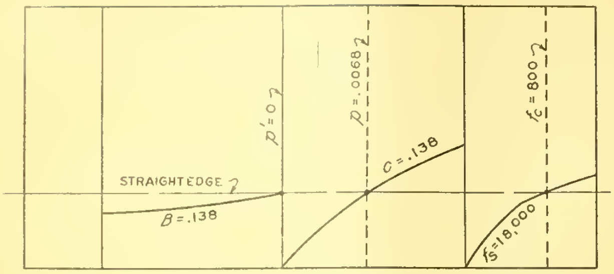


Figure 2.—Diagram for Use with Example 1.

the straightedge to the intersection with $C=.138$, and read vertically downward $p=.0068$.

From the formula for B for simple bending, given on each diagram, $B=\frac{M}{f_c bd^2}$ we obtain:

$$bd^2=\frac{M}{Bf_c}=\frac{1{,}000{,}000}{.138\times 800}=9{,}050$$

The section may now be proportioned as desired, and the area of reinforcement obtained by multiplying the value of effective area of the section, *bd*, by the value of p.

Example 2.—Simple Bending With Tensile Reinforcement Only; Analysis.—Given: A member 15 inches wide and 27 inches deep is subjected to a bending moment of 900,000 inch-pounds. Reinforcement consists of two 1-inch square bars. The distance from face of concrete to centerline of steel is 2 inches. Find f_c and f_s.

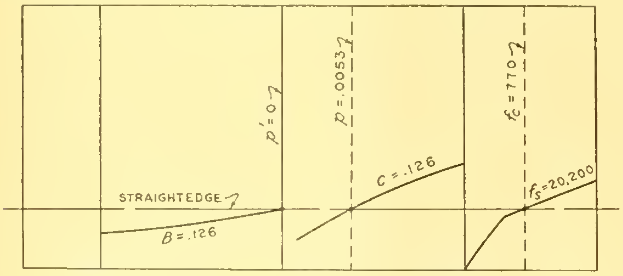


Figure 3.—Diagram for Use with Example 2.

Solution: Select any diagram. From the given data, $p'=0$ and $p=.0053$. For simple bending, $B=C$. Shift the straightedge along the vertical lines representing the values of p' and p, and, by trial, locate the position at which the value of B equals the value of C. This fulfills the condition that the ratio $\frac{B}{C}=1$ for simple bending. Using the given data, the value of B at this position is found to be .126. From the expression $B=\frac{M}{f_c bd^2}$ for simple bending, we obtain:

$$f_c=\frac{M}{Bbd^2}=\frac{900{,}000}{.126\times 15\times 25^2}=770 \text{ pounds per square inch.}$$

With the straightedge held in its last position note that it intersects the vertical line representing $f_c=770$ pounds per square inch at $f_s=20{,}200$ pounds per square inch.

Example 3.—Simple Bending With Tensile and Compressive Reinforcement; Design.—Given: A member 10 inches wide and 18 inches deep is subjected to a bending moment of 363,000 inch-pounds. The distance from face of concrete to centerline of steel is to be 3 inches. Allowable stresses, $f_c=850$ pounds per square inch, and $f_s=16{,}000$ pounds per square inch. Design the reinforcement.

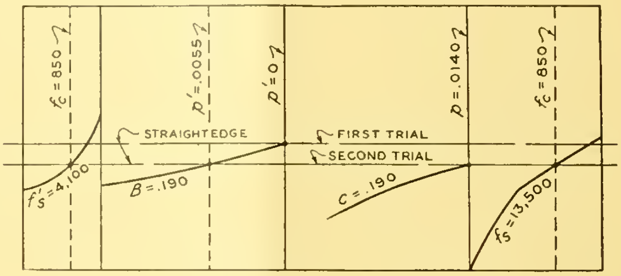


Figure 4.—Diagram for Use with Example 3.

Solution: $\frac{d'}{d}=\frac{3}{18-3}=0.20$. Therefore, use the diagram based on this value.

$$B=\frac{M}{f_c bd^2}\text{ (for simple bending)}=\frac{363{,}000}{850\times 10\times 15^2}=0.190$$

$$C=B\text{ (for simple bending)}=0.190$$

First investigate the section for tensile reinforcement only by placing the straightedge at the intersection of lines corresponding to $B=0.190$ and $p'=0$. Follow back along the straightedge to the intersection with the curve for $C=0.190$ and read vertically downward to obtain the value of p. It is evident that the value of p will be considerably in excess of .0140 as the required intersection lies well outside the limit of the curves for C. Now shift the straightedge downward to the intersection, say, of lines corresponding to $C=0.190$ and $p=.0140$. At the intersection with the curve $B=0.190$ read $p'=.0055$, and at the intersections with the ordinates representing $f_c=850$ pounds per square inch read $f_s=13{,}500$ pounds per square inch and $f'_s=4{,}100$ pounds per square inch.

By thus shifting the straightedge up or down, similar consistent values of p, p', f_s and f'_s may be obtained by inspection.

Example 4.—Simple Bending With Tensile And Compressive Reinforcement; Analysis.—Given: A member 12 inches wide and 15 inches deep is subjected to a bending moment of 300,000 inch-pounds. Reinforcement consists of two 1-inch square bars in tension and one 1-inch square bar in compression. The distance from face of concrete to centerline of steel is 3 inches. Find f_c, f_s and f'_s.

Solution: $\frac{d'}{d}=\frac{3}{15-3}=0.25$. Therefore use the diagram based on this value. From the given data, $p=.0139$ and $p'=.0070$. For simple bending, $\frac{B}{C}=1$. Shift the straightedge along the vertical lines representing the above values of p and p' and, by trial, locate the position at which $B=C$. The value of B at this

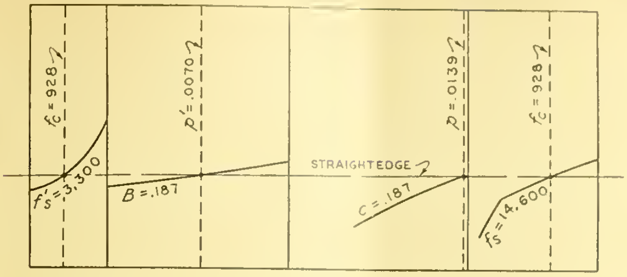


Figure 5.—Diagram for Use with Example 4.

position is found to be 0.187. From the expression for simple bending, $B=\frac{M}{f_c bd^2}$ we obtain

$$f_c=\frac{M}{Bbd^2}=\frac{300,000}{.187\times12\times12^2}=928 \text{ pounds per square inch.}$$

With the position of the straightedge unchanged, at the intersection with the vertical lines representing $f_c=928$ pounds per square inch, read $f_s=14,600$ pounds per square inch and $f'_s=3,300$ pounds per square inch.

Example 5.—Bending and Direct Stress With Tensile Reinforcement Only; Design.—Given: A member 12 inches wide and 15 inches deep is subjected to a bending moment of 161,000 inch-pounds and a normal force of 7,160 pounds. The distance from face of concrete to centerline of steel is 3 inches. Allowable stresses, $f_c=800$ pounds per square inch and $f_s=18,000$ pounds per square inch. Design for tensile reinforcement only.

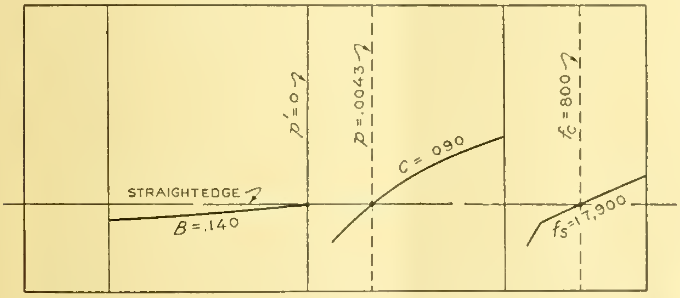


Figure 6.—Diagram for Use with Example 5.

Solution: Any one of the diagrams may be used. For this solution, the diagram based on $\frac{d'}{d}=0.20$ is selected.

From the given data, $e=\frac{M}{N}=22.5$ inches, $d=12$ inches and $e'=27$ inches.

$$B=\frac{Ne'}{f_c bd^2}=\frac{7,160\times27}{800\times12\times12^2}=0.140$$

$$C=B\left(\frac{e'-0.80d}{e'}\right)=0.140\left(\frac{27-9.6}{27}\right)=0.090$$

Place a straightedge on the diagram passing through the intersection of lines representing $p'=0$ and $B=.140$. Follow back along the straightedge to the intersection with the curve for $C=0.090$, and read vertically downward to obtain $p=.0043$. At the intersection with the ordinate for $f_c=800$ pounds per square inch read $f_s=17,900$ pounds per square inch.

Example 6.—Bending and Direct Stress With Tensile Reinforcement Only; Analysis.—Given: A member 15 inches wide and 22 inches deep is subjected to a bending moment of 500,000 inch-pounds and a normal force of 15,150 pounds. Reinforcement consists of 2 1-inch square bars in tension. The distance from face of concrete to centerline of steel is 2 inches. Find f_c and f_s.

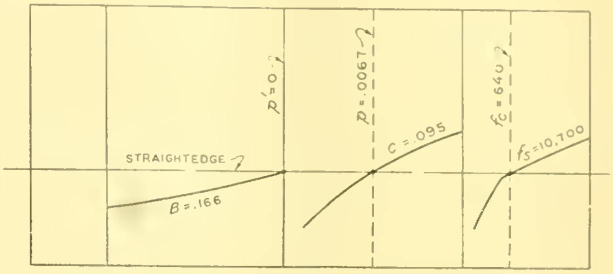


Figure 7.—Diagram for Use with Example 6.

Solution: Any one of the diagrams may be used. For this solution, the diagram based on $\frac{d'}{d}=.10$ is selected.

From the given data, $e=\frac{M}{N}=33$ inches, $d=20$ inches, $e'=42$ inches, $p'=0$ and $p=.0067$.

$$C=B\left(\frac{e'-0.90d}{e'}\right), \text{ whence } \frac{B}{C}=\frac{e'}{e'-0.90d}=\frac{42}{42-18}=1.75$$

Shift a straightedge along the two vertical lines corresponding to $p'=0$ and $p=.0067$, and, by trial, locate the position at which $\frac{B}{C}=1.75$, the computed value.

The value of B at this position is found to be 0.166.

Using this value in the equation, $B=\frac{Ne'}{f_c bd^2}$ we have:

$$f_c=\frac{Ne'}{Bbd^2}=\frac{15,150\times42}{.166\times15\times20^2}=640 \text{ pounds per square inch.}$$

With the straightedge in this position, at the intersection with the ordinate for $f_c=640$ pounds per square inch read $f_s=10,700$ pounds per square inch.

Example 7.—Bending And Direct Stress With Tensile And Compressive Reinforcement; Design.—Given: A member 12 inches wide and 18 inches deep is subjected to a bending moment of 550,000 inch-pounds and a normal force of 30,000 pounds. The distance from face of concrete to centerline of steel is 3 inches. Allowable stresses, $f_c=1,200$ pounds per square inch and $f_s=20,000$ pounds per square inch. Design the reinforcement.

Solution: From the given data, $\frac{d'}{d}=0.20$, $d=15$ inches, $e=\frac{M}{N}=18.3$ inches, and $e'=24.3$ inches.

$$B=\frac{Ne'}{f_c bd^2}=\frac{30,000\times24.3}{1200\times12\times15^2}=0.225$$

$$C=B\left(\frac{e'-0.80d}{e'}\right)=0.225\left(\frac{24.3-12}{24.3}\right)=0.114$$

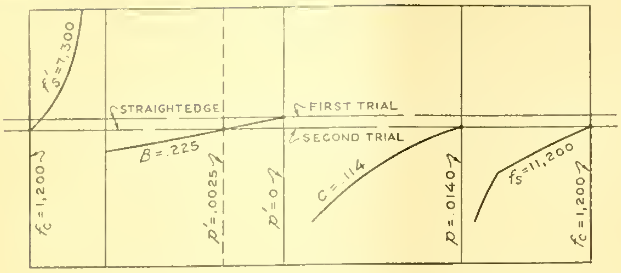


Figure 8.—Diagram for Use with Example 7.

First investigate the section for tensile reinforcement only by placing the straightedge at the intersection of the curve for $B=0.225$ and the ordinate $p'=0$. Follow back along the straightedge to the intersection with the curve for $C=0.114$, reading vertically downward to obtain the value of p. The required intersection lies outside the limits of the C-curves, indicating an excessive amount of tensile reinforcement. Shift the straightedge down to the intersection, say, of the curve for $C=0.114$ and the ordinate for $p=.0140$. At the intersection with the curve for $B=0.225$, read $p'=.0025$, and at the intersections with the ordinates for $f_c=1,200$ pounds per square inch read $f_s=11,200$ pounds per square inch and $f'_s=7,300$ pounds per square inch.

By thus shifting the straightedge up and down, similar consistent values of p, p', f_s and f'_s may be read by inspection, and the desired combination selected.

Example 8.—Bending and Direct Stress With Tensile And Compressive Reinforcement; Analysis.—Given: A member 12 inches wide and 24 inches deep is subjected to a bending moment of 600,000 inch-pounds and a normal force of 75,000 pounds. Reinforcement consists of four ⅞-inch round bars in tension and two ⅞-inch round bars in compression. The distance from face of concrete to centerline of steel is 3 inches. Find f_c, f_s and f'_s.

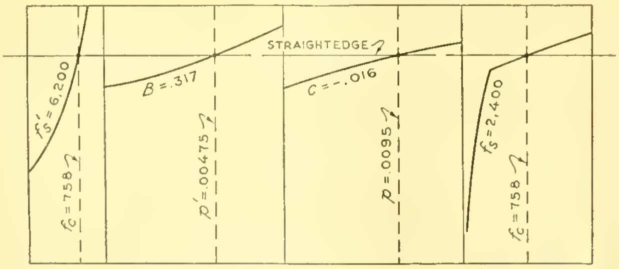


Figure 9.—Diagram for Use with Example 8.

Solution: From the given data, $e=\frac{M}{N}=8$ inches, $e'=17$ inches, $d=21$ inches, $p'=.00475$ and $p=.0095$. $\frac{d'}{d}=0.15$.

$$\frac{B}{C}=\frac{e'}{e'-0.85d}=\frac{17}{17-17.85}=-20.0$$

Shift a straightedge along the two ordinates representing $p'=.00475$ and $p=.0095$, and, by trial, locate the position at which $\frac{B}{C}=-20.0$

The value of B at this position is found to be 0.317

Using this value in the equation $B=\frac{Ne'}{f_cbd^2}$ we have:

$$f_c=\frac{Ne'}{Bbd^2}=\frac{75,000\times17}{.317\times12\times21^2}=758 \text{ pounds per square inch.}$$

With the straightedge in this position, at the intersections with the ordinates for $f_c=758$ pounds per square inch, read $f_s=2,400$ pounds per square inch and $f'_s=6,200$ pounds per square inch.

Example 9.—Bending and Direct Stress With Tensile Reinforcement Only; Steel Controls.—Given: A member 30 inches wide and 70 inches deep is subjected to a bending moment of 876,765 foot-pounds and a normal force of 91,850 pounds. The distance from face of concrete to centerline of steel is 3 inches. Allowable stresses, $f_c=800$ pounds per square inch and $f_s=18,000$ pounds per square inch. Design for tensile reinforcement only.

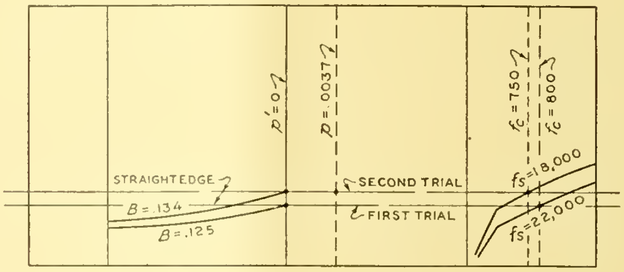


Figure 10. Diagram for Use with Example 9.

Solution: Any one of the diagrams may be used. For this solution, the diagram based on $\frac{d'}{d}=0.05$ is selected. From the given data, $e=\frac{M}{N}=\frac{876,765}{91,850}\times12=115$ inches, $d=67$ inches, $N=91,850$, and $e'=147$ inches.

First compute B on the slide rule in such manner that the last setting is division by f_c. Place the straightedge so it will pass through the point on the curve for the value of B where the curve is intersected by the ordinate for $p'=0$, and check f_s as indicated by the intersection of the straightedge with the ordinate for $f_c=800$. If f_s is found to be excessive change the slide rule setting to a lower value of f_c and again determine the value of f_s using the ordinate for the new value of f_c. When a value of f_c giving a satisfactory value of f_s is determined, compute C and then read the value of p corresponding to the intersection of the straightedge with the curve for C.

First assumption: $f_c=800$ pounds per square inch.;

$$=\frac{Ne'}{f_cbd^2}=\frac{91,850\times147}{800\times30\times67^2}=.125;\ f_s=22,000 \text{ pounds per square inch.}$$

Second assumption: $f_c=750$ pounds per square inch.;
$B=.134$ $f_s=18,000$ pounds per square inch.

$$C=B\left(\frac{e'-0.95d}{e'}\right)=.134\left(\frac{147-64}{147}\right)=.076$$

At the intersection of the straightedge in its final position with the curve $C=.076$, read $p=.0037$.

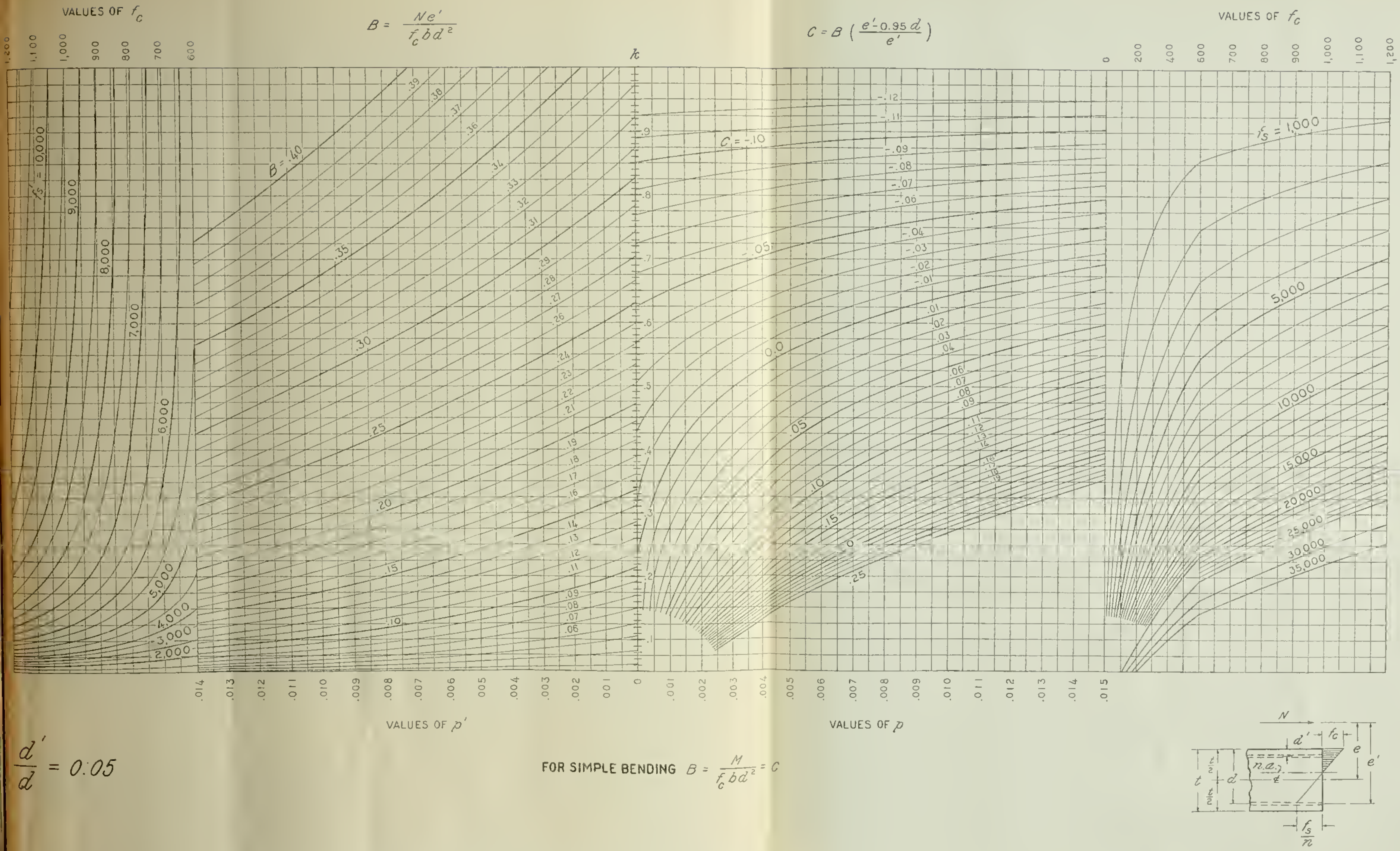


PLATE 1.—DIAGRAM FOR DESIGN AND ANALYSIS OF RECTANGULAR SECTIONS OF REINFORCED CONCRETE BASED ON $n = 10$ AND $\frac{d'}{d} = 0.05$.

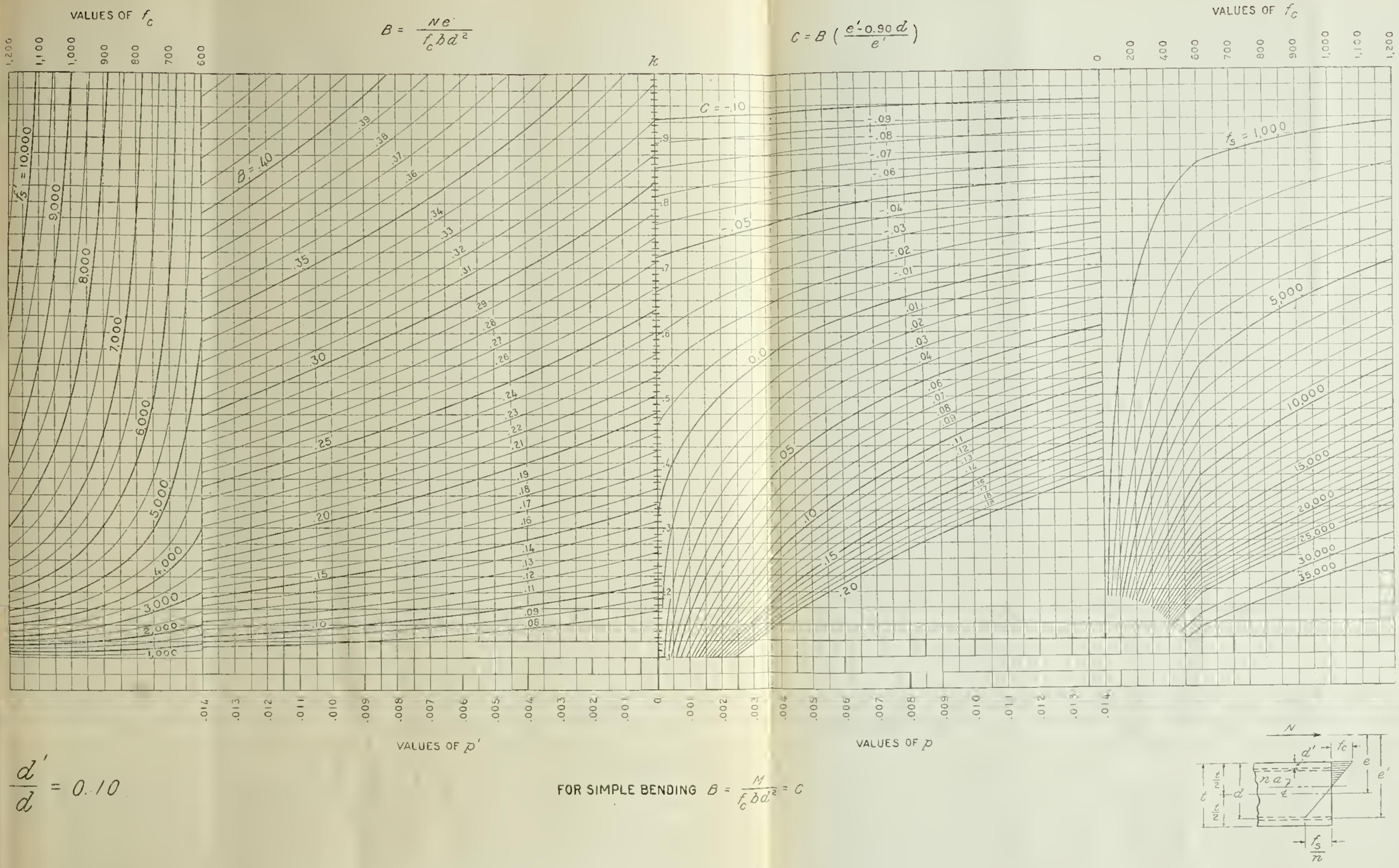


Plate 2.—Diagram for Design and Analysis of Rectangular Sections of Reinforced Concrete Based on $n = 10$ and $\frac{d'}{d} = 0.10$.

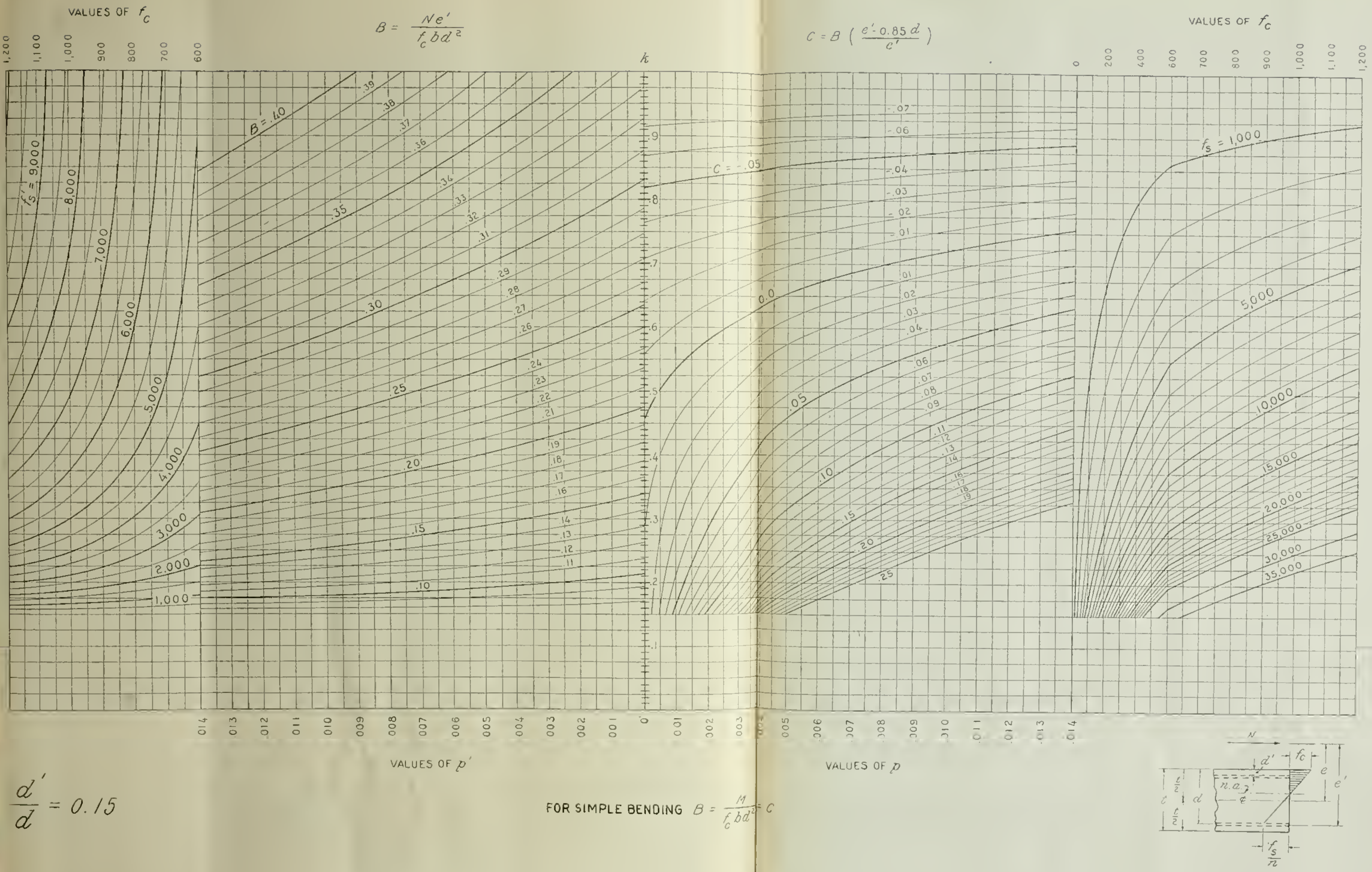
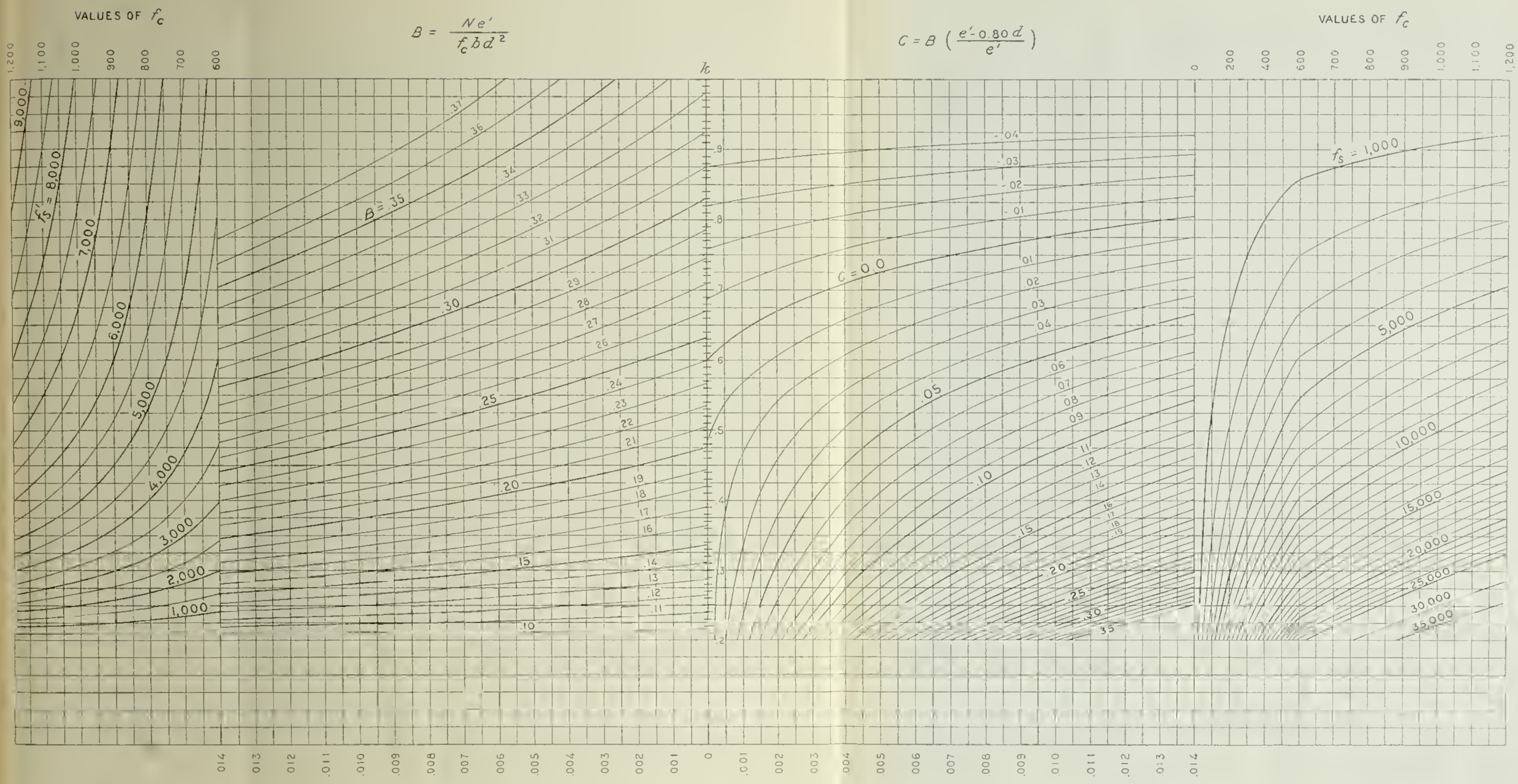


PLATE 3.—DIAGRAM FOR DESIGN AND ANALYSIS OF RECTANGULAR SECTIONS OF REINFORCED CONCRETE BASED ON $n = 10$ AND $\frac{d'}{d} = 0.15$.



$$\frac{d'}{d} = 0.20$$

FOR SIMPLE BENDING $B = \frac{M}{f_c b d^2} = C$

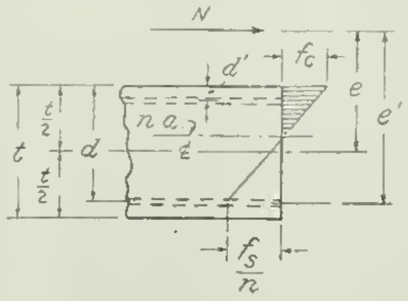


PLATE 4.—DIAGRAM FOR DESIGN AND ANALYSIS OF RECTANGULAR SECTIONS OF REINFORCED CONCRETE BASED ON $n = 10$ AND $\frac{d'}{d} = 0.20$.

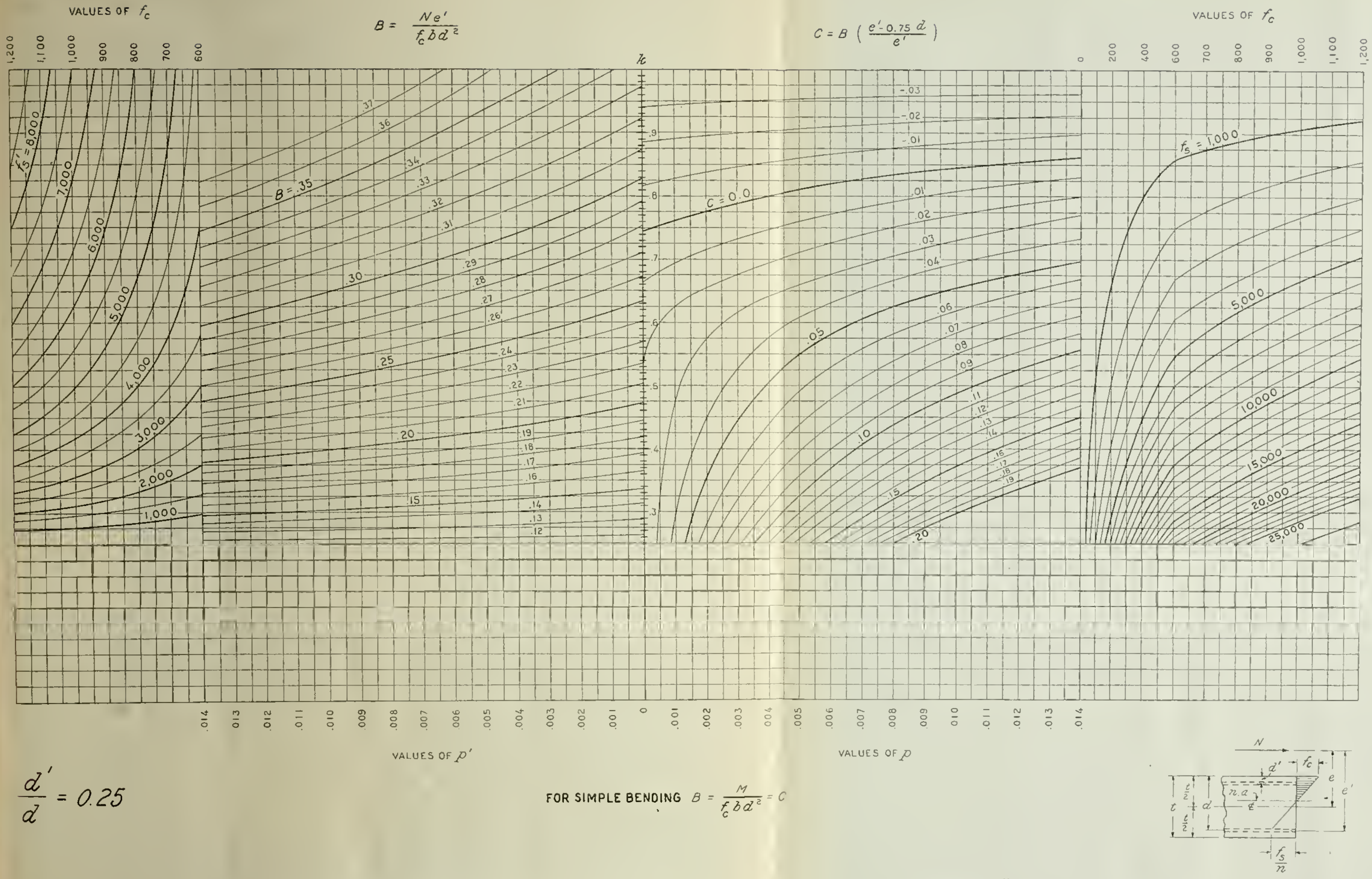


Plate 5.—Diagram for Design and Analysis of Rectangular Sections of Reinforced Concrete Based on $n=10$ and $\frac{d'}{d}=0.25$.

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A CONTINUOUS CONCRETE BRIDGE

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Volume 23, No. 11 January–February–March 1944

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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COMPUTING MOMENTS FOR CONTINUOUS CONCRETE BRIDGES

A METHOD APPLICABLE TO THREE SYMMETRICAL SPANS SIMPLY SUPPORTED

Reported by A. L. **GOOGINS**, Highway Bridge Engineer, G. B. **CANAGA**, Highway Bridge Engineer,
and D. P. **BABCOCK**, Highway Bridge Engineer

A WELL DESIGNED continuous bridge has a depth of section that varies closely with the bending moment requirements from a minimum at the center of spans to a maximum at the intermediate supports. Reduction in deck depth, particularly at midspan, gives the continuous bridge an important economic advantage and improves architectural appearance. Only single bearings are required at interior supports, materially reducing the width of piers below that required for simple spans.

The time and difficulties involved in the design of continuous bridges, with members of variable moment of inertia, have prevented some designers from adopting such a design where it was a logical choice.

Continuous bridges may be of several spans of varying length but the more common type has three symmetrical, simply-supported spans. The charts accompanying this article have been prepared to simplify the design of this type of bridge.[1]

PROCEDURE IN PLOTTING CURVES

The curves presented here are plotted for continuous slab bridges but may be used in designing continuous girder bridges by transforming the girder section into a slab section having equivalent moments of inertia and using the value of r as determined for the transformed slab.

The terms used in this article are as follows:

See figure 2.

hc = depth of transformed slab at center of span,
r = ratio of increased depth, of transformed slab, at interior supports to depth of slab at center of span,
l = length of end spans,
n = ratio of length of center span to length of end span.

See figure 3.

y = difference between depth of girder at center of span and depth of girder at interior supports,
$x = \frac{\text{span length}}{2}$.

See figures 8 and 9.

K = ratio of distance from left end of span to position of concentrated load on the span, to the length of span.

See figure 10.

t = thickness of slab,
h = depth of girder section,
b = width of girder flange,
b_1 = width of girder stem.

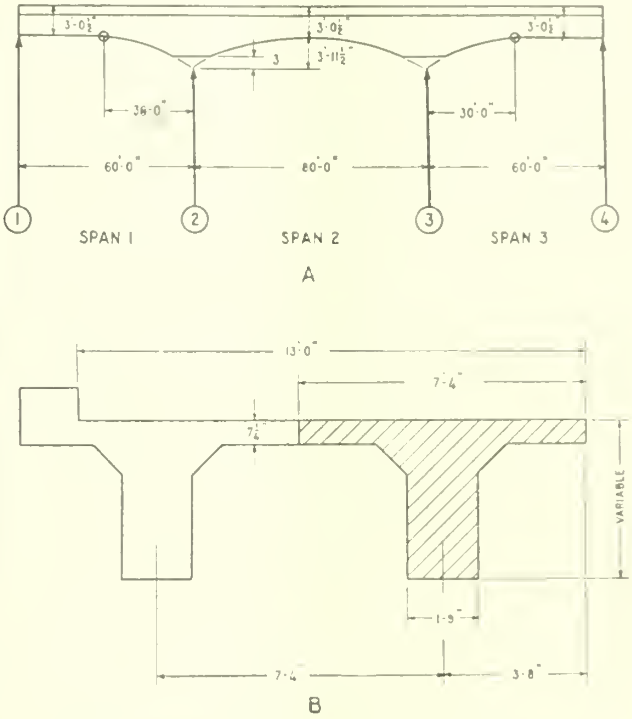

Figure 1.—Girder Elevation and Cross Section.

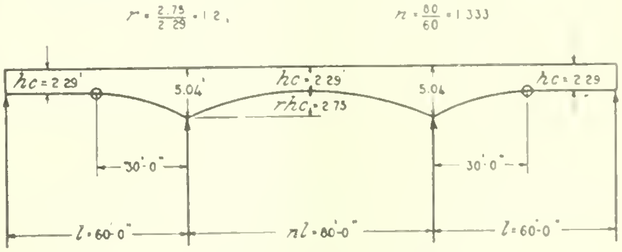

Figure 2.—Dimensions of Slab as Determined From the Transformed Girder.

The coefficient values used in plotting the curves of figures 5 to 9 were obtained by first determining the moments at the supports, by the distribution of fixed end moments.

When produced by a concentrated load these moments are influenced by the position of the load on the spans, by the length of the center span in relation to the length of the end span and by the relative thickness of the slab. These three variables, represented by K, n, and r require that two coefficients c and c_1 be used in expressing the moments in terms of Pl.

The moments at the supports in terms of Pl were first determined for various values of K and r when $n = 1.25$

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[1] The publication "Continuous Concrete Bridges" (second edition), published by the Portland Cement Association will be found very useful in designing continuous concrete bridges having more than three spans and in designing unsymmetrical bridges.

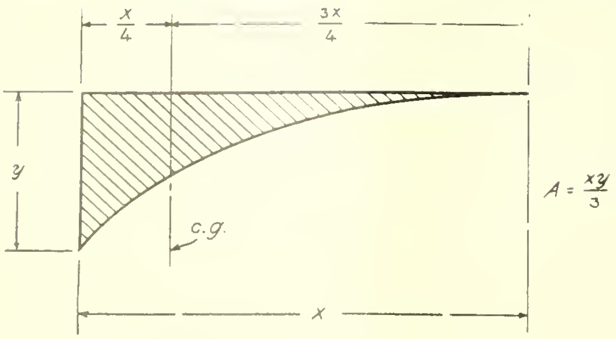


FIGURE 3. COMPLEMENT OF HALF PARABOLA.

The moments thus determined were equated to cPl and the coefficients c determined. The moments at the supports in terms of Pl were then determined for other values of n, letting n vary from values of 1 to 2. The coefficients, c_1, express the moments for values of n other than 1.25 in percentage of the values as determined when $n=1.25$.

For a uniform load on any span fully loaded, the variable K does not exist, which permits the use of only one coefficient, c, in expressing the moments at the supports in terms of wl^2.

The stiffness factors, carry-over factors, and fixed end moments, necessary for the determination of the true moments at the supports by moment distribution, may be computed or taken from various tables prepared and published in connection with the subject of moment distribution. In this analysis the tables in the bulletin "Continuous Concrete Bridges" (second edition) published by the Portland Cement Association, were used.

If a continuous concrete slab is to be designed, it is first necessary to assume a depth of slab at the center of spans and a depth at the supports, from which the constants required in using the charts may be computed. Knowing the dead load of the assumed slab, the live load to be carried and the constants determined from the slab shape, it is possible to compute the moments and unit stresses for the assumed slab and revise the slab depth as the unit stresses indicate.

PROCEDURE IN DESIGNING A REINFORCED CONCRETE GIRDER BRIDGE

In designing a continuous reinforced concrete girder bridge, the width of roadway, the span arrangement, and the live load to be carried, will be known. It is necessary to assume the number, size, and spacing of the girders. The floor slab can then be designed and the loads to be carried by each of the trial girders determined. The next step is to transform the assumed girder into a slab shape having corresponding moments of inertia. With the structure constant, r, as determined for the transformed slab, together with the loads as computed for the assumed girder, the charts may be used to compute the girder moments. If the moments thus determined give unsatisfactory unit stresses, the girder shape must be changed and the process repeated until the desired unit stresses are obtained. Much work may be saved in determining trial sizes by using section 6 of "Continuous Concrete Bridges" published by the Portland Cement Association.

The example which follows is not a complete girder design, but is given to show the use of the charts in computing the moments, which when known, make stress determination a simple process.

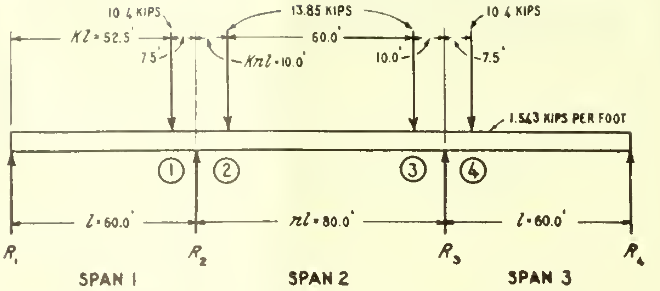


FIGURE 4. AMOUNT AND POSITION OF GIRDER DEAD LOADS AS COMPUTED.

Assume three continuous spans as shown in figure 1. Transform the girder shown shaded in figure 1,B into a slab having equivalent moments of inertia.

The moments of inertia of the girder sections and for the equivalent slab sections are computed for the gross concrete sections without reinforcement.

Use figure 10 and compute moments of inertia for the girder at supports (2) and (3) figure 1 and at the center of the spans.

At supports:

$t=7\frac{1}{4}''$ (fig. 1, B),
$h=7'\ 0''$ (fig. 1, A),
$b=7'\ 4''$ (fig. 1, B),
$b_1=1'\ 9''$ (fig. 1, B).

$$\frac{t}{h}=\frac{0.604}{7.0}=0.0863$$

$$\frac{b_1}{b}=\frac{1.75}{7.33}=0.2387$$

In figure 10, the ordinate for $\frac{t}{h}=0.0863$, intersects the curve for $\frac{b_1}{b}=0.2387$ (visualized by interpolation) at $c=0.0312$. The moment of inertia, I, of the gross girder section at the supports, neglecting reinforcement, is $I=cbh^3=0.0312\times7.33\times7.0^3=78.4$ feet^4.

The depth at the supports for an equivalent slab section can then be computed by the formula $\frac{bd^3}{12}=78.4$ and solving for d. Here b, has the same value as before.

$$\frac{7.33d^3}{12}=78.4$$

$$d=\sqrt[3]{\frac{12\times78.4}{7.33}}=5.04\text{ feet.}$$

At the center of spans:

$t=7\frac{1}{4}''$ (fig. 1, B),
$h=3'\ \frac{1}{2}''$ (fig. 1, A),
$b=7'\ 4''$ (fig. 1, B),
$b_1=1'\ 9''$ (fig. 1, B).

$$\frac{t}{h}=\frac{0.604}{3.04}=0.1988$$

$$\frac{b_1}{b}=\frac{1.75}{7.33}=0.2387$$

In figure 10 these values of $\frac{t}{h}$ and $\frac{b_1}{b}$ give the value $c=0.0354$. The moment of inertia, I, of the gross girder section at the center of the spans, neglecting reinforcement, is

$$I=cbh^3=0.0354\times7.33\times3.04^3=7.3\text{ feet}^4$$

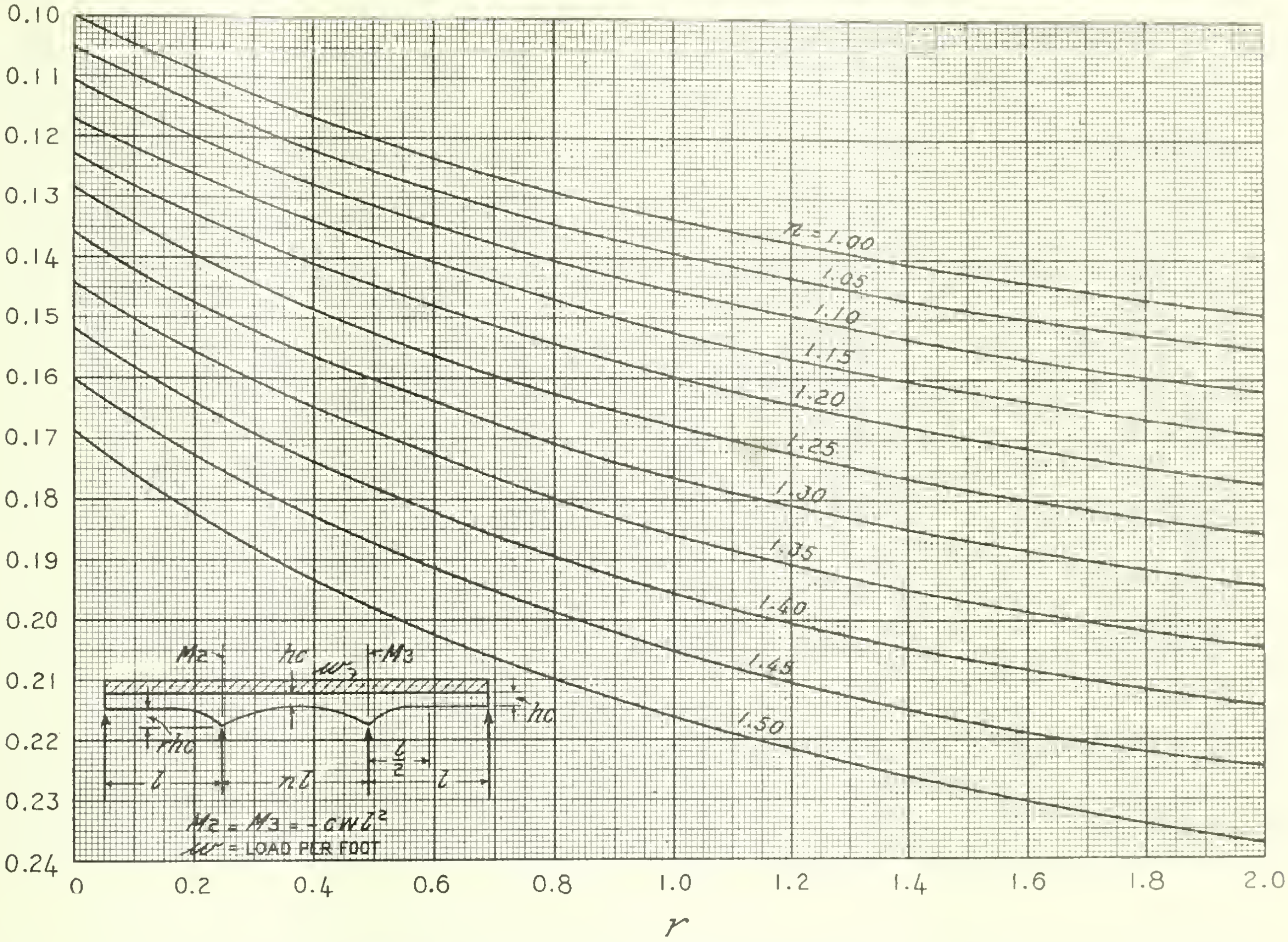


FIGURE 5.—DIAGRAM FOR DETERMINING MOMENTS AT INTERIOR SUPPORTS PRODUCED BY UNIFORM LOAD ON ALL SPANS.

The depth of an equivalent slab section, at the center of spans may now be computed:

$$\frac{bd^3}{12}=7.3$$

$$\frac{7.33d^3}{12}=7.3$$

$$d=\sqrt[3]{\frac{12\times7.3}{7.33}}=2.29 \text{ feet.}$$

Figure 2 shows the equivalent slab from which the values of r and n are determined.

For use in the solution with the charts $hc=2.29$; $rhc=5.04-2.29=2.75$; $r=\frac{2.75}{2.29}=1.2$; and $n=\frac{80}{60}=1.333$.

Compute the dead loads carried by the interior girder shown in figure 1, considering the weight of the curved portion of the girder stem as a concentrated dead load placed at the center of gravity of the complement of a half parabola. This may be done without changing the final solution appreciably, because the moments produced by the weight of the curved portion of the stem are very small in comparison to the total moments produced. The properties of the complement of a half parabola, are shown in figure 3. The remaining portion of the dead load is assumed uniform over all spans.

For dead loads considered as uniform:

$7.33\times0.604\times0.15=0.664$ kips, weight of slab,
$1.75\times2.436\times0.15=0.640$ kips, weight of girder stem,
$0.5\times0.75\times0.15=0.056$ kips, weight of girder fillets,
$7.33\times0.025=0.183$ kips, future wearing surface,
1.543 kips, uniform dead load per foot of girder.

For dead loads considered as concentrated in the end spans:

$y=3.96$ feet, $x=30.0$ feet, $b_1=1.75$ feet.

$\frac{3.96\times30}{3}\times1.75\times0.15=10.4$ kips, weight of curved portion of stem (spans 1 and 3).

$\frac{30}{4}=7.5$ feet, distance of load from intermediate support (spans 1 and 3).

For dead loads considered as concentrated in the center span:

$y=3.96$ feet, $x=40.0$ feet; $\frac{3.96\times40}{3}\times1.75\times0.15$ $=13.85$ kips (span 2).

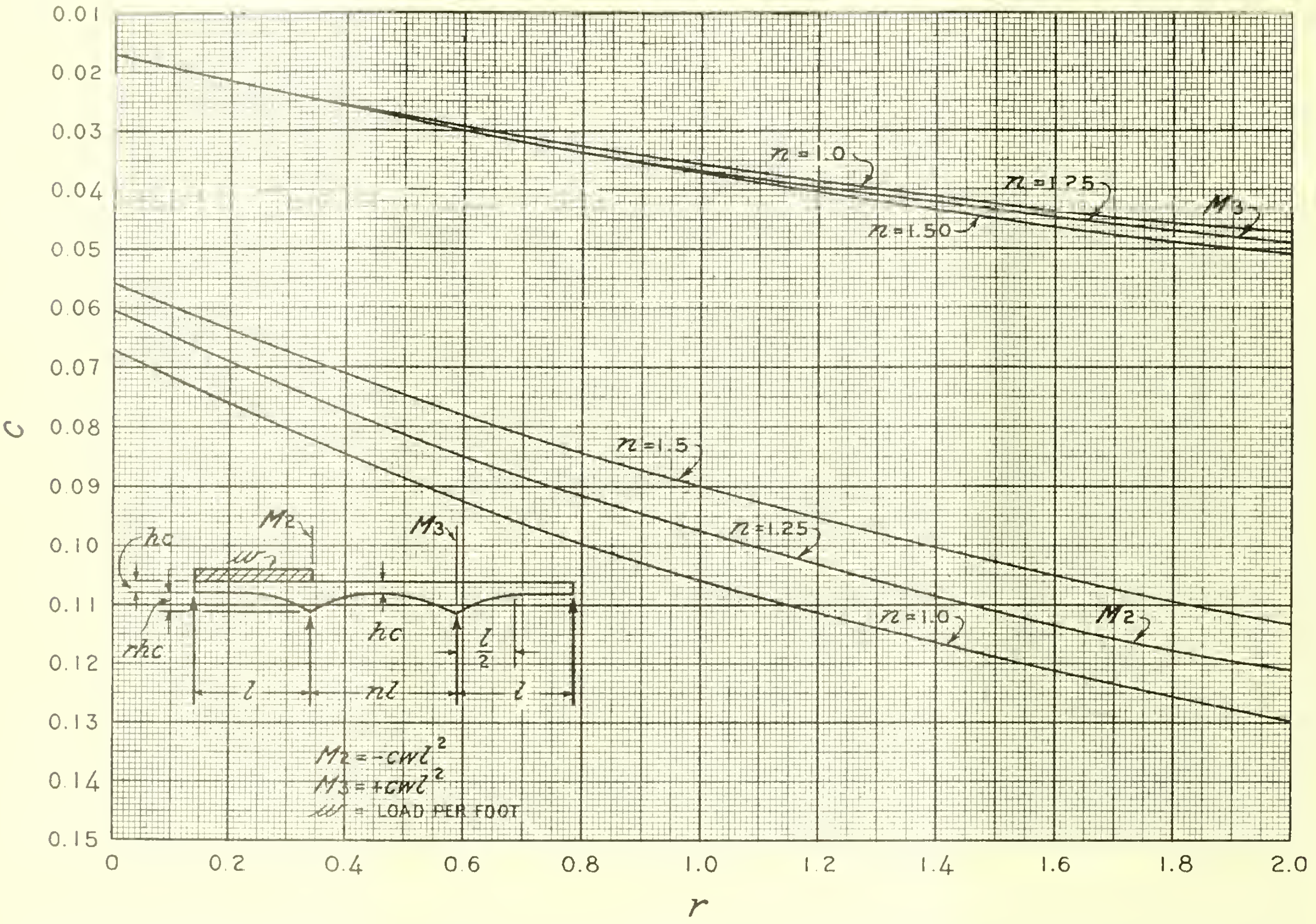


Figure 6.—Diagram for Determining Moments at Interior Supports Produced by Uniform Load on One End Span.

$\frac{40}{4}=10.0$ feet, distance of concentrated loads, in span 2, from supports.

Figure 4 shows the girder dead loads as computed above. The numbers in circles are for reference later in the example.

Consider that the bridge is to be designed for the equivalent H–20 live load as given by the Standard Specifications for Highway Bridges adopted by the American Association of State Highway Officials 1941. For this example we will use an impact factor of 0.27.

The lane width by the above specifications is to be 10 feet. Since the flange of the girder which we are considering is 7 feet, 4 inches wide, let it be assumed that the girder carries 0.733 of one lane load.

$0.733 \times 0.64=0.469$ kips per foot, uniform live load.

$0.733 \times 18.0=13.2$ kips, concentrated live load for moment.

In the determination of stress and the proper arrangement of reinforcing steel, it is necessary to know the moment and shears at many points. Much work can be saved, however, if in the preliminary calculations only the maximum negative moments at the supports and the maximum positive moments in the spans are determined.

COMPUTATION OF DEAD LOAD MOMENTS AT R_2

The positions of the dead loads are shown in figure 4.

Uniform dead load.—From figure 2, $r=1.2$, $n=1.333$, and from figure 5, $c=0.188$.

$M_2=-cwl^2=-0.188 \times 1.543 \times 60^2=-1044.3$ kip-feet

Concentrated dead loads.—For load (1)=10.4 kips, use figure 8. In addition to the values of n and r already determined from figure 2, a value of K, indicating the position of the load on the span, is required. (See fig. 4.)

$$K=\frac{52.5}{60}=0.875 \text{ for load (1).}$$

In figure 8 the ordinate for $K=0.875$ intersects the curve for $r=1.2$ (in group of curves for M_2) at a point $c=0.065$. Also from the small diagram at the lower right of figure 8 with $n=1.333$ and $r=1.2$ we obtain $c_1=0.975$. The moment at R_2 due to load (1) is then:

$M_2=-cc_1\ Pl=-0.065 \times 0.975 \times 10.4 \times 60=-39.5$ kip-feet.

For load (2) use figure 9.

$P=13.85$; $l=60.0$ feet; $K=0.125$; $r=1.2$; $n=1.333$

For moment at R_2, $c=0.0665$; $c_1=1.09$.

$M_2=-cc_1\ Pl=-0.0665 \times 1.09 \times 13.85 \times 60=-60.2$ kip-feet.

For load (3) use figure 9.

$$Knl=70 \text{ feet; } nl=80 \text{ feet; } K=\frac{70}{80}=0.875.$$

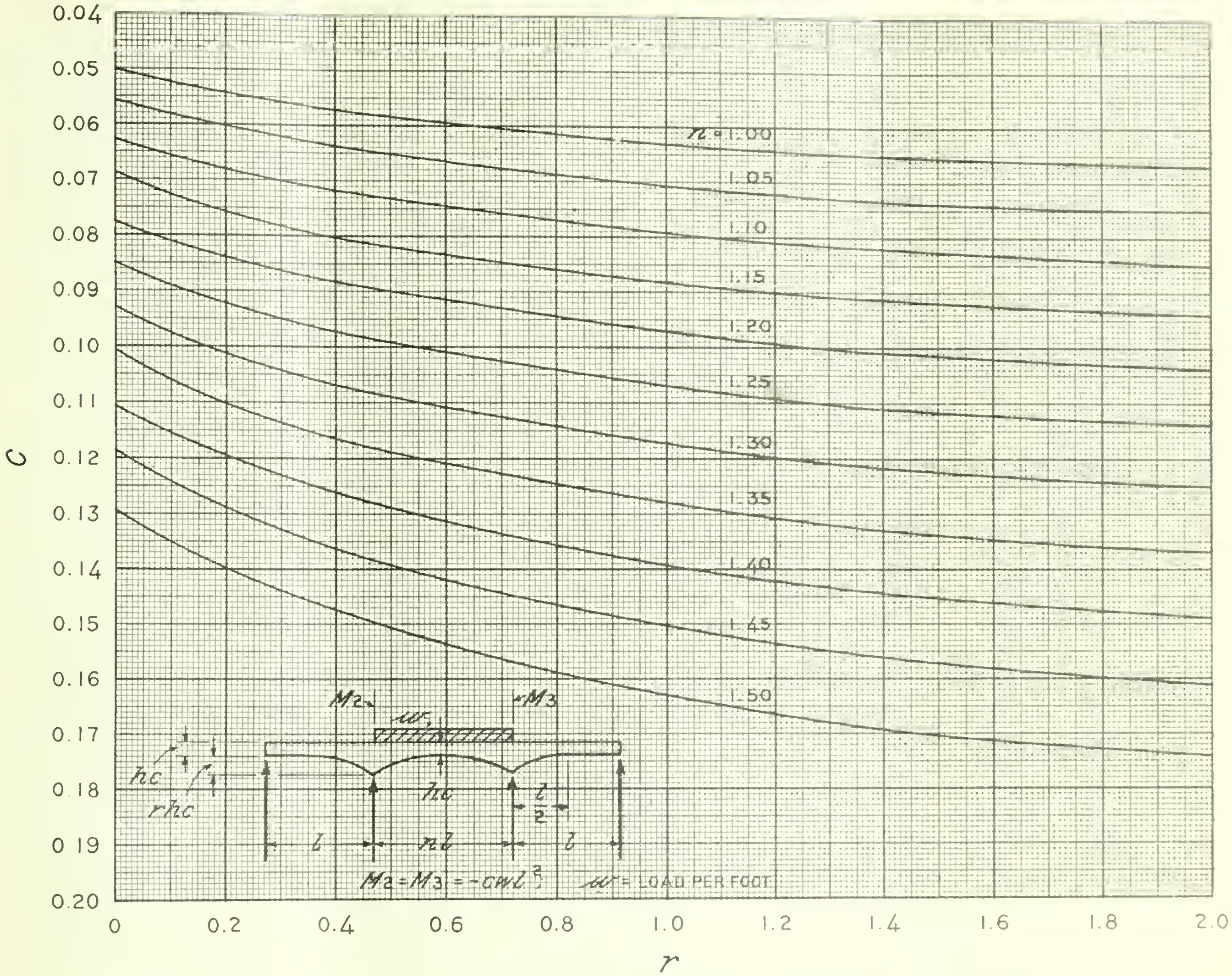


Figure 7.—Diagram for Determining Moments at Interior Supports Produced by Uniform Load on Center Span.

$P = 13.85$ kips; $l = 60$ feet; $r = 1.2$; $n = 1.333$

For moment at R_2, $c = 0.0345$; $c_1 = 1.09$.

$M_2 = -cc_1\ Pl = -0.0345 \times 1.09 \times 13.85 \times 60 = -31.3$ kip-feet

For load (4) use figure 8.

Since the spans are symmetrical the moment at R_2 caused by load (4) will be the same as the moment at R_3 due to load (1). From figure 8 compute M_3 due to load (1).

$K = 0.875$; $r = 1.2$; $n = 1.333$; $P = 10.4$ kips

$l = 60$ feet; $c = 0.0245$; $c_1 = 1.0$

$M_2 = +cc_1\ Pl = +0.0245 \times 1.0 \times 10.4 \times 60 = +15.3$ kip-feet

Summary of dead load moments at R_2

−1,044.3 kip-feet, due to uniform, dead load
− 39.5 kip-feet, due to load (1),
− 60.2 kip-feet, due to load (2),
− 31.3 kip-feet, due to load (3),
+ 15.3 kip feet, due to load (4),

−1,160.0 kip-feet, total dead load moment at R_2.

Since the spans are symmetrical and the dead loads are symmetrically placed on the spans, the moments at R_2 and R_3 will be equal.

COMPUTATION OF LIVE LOAD MOMENTS AT R.

Uniform live load.—The maximum negative moment at R_2 for the uniform live load occurs when spans 1 and 2 are fully loaded, with no load on span 3.

Use figure 6 to find the moment at R_2 due to the uniform live load on span 1.

$w = 0.469$ kips per foot; $l = 60$ feet

$r = 1.2$; $n = 1.333$; $c = 0.101$

$M_2 = -cwl^2 = -0.101 \times 0.469 \times 60^2 = -170.5$ kip-feet

Use figure 7 to find the moment at R_2 due to the uniform live load on span 2.

$w = 0.469$; $l = 60$; $r = 1.2$; $n = 1.333$

$c = 0.128$.

$M_2 = -0.128 \times 0.469 \times 60^2 = -216.1$ kip-feet

The moment at R_2 with both spans loaded is then $-170.5 - 216.1 = -386.6$ kip-feet.

Concentrated live loads.—Figures 8 and 9 are used for finding moments due to concentrated loads.

Use figure 8 and compute M_2, when $P = 13.2$; $r = 1.2$; $n = 1.333$.

$K = 0.52$ (inspect chart and select value of K corresponding to maximum moment).

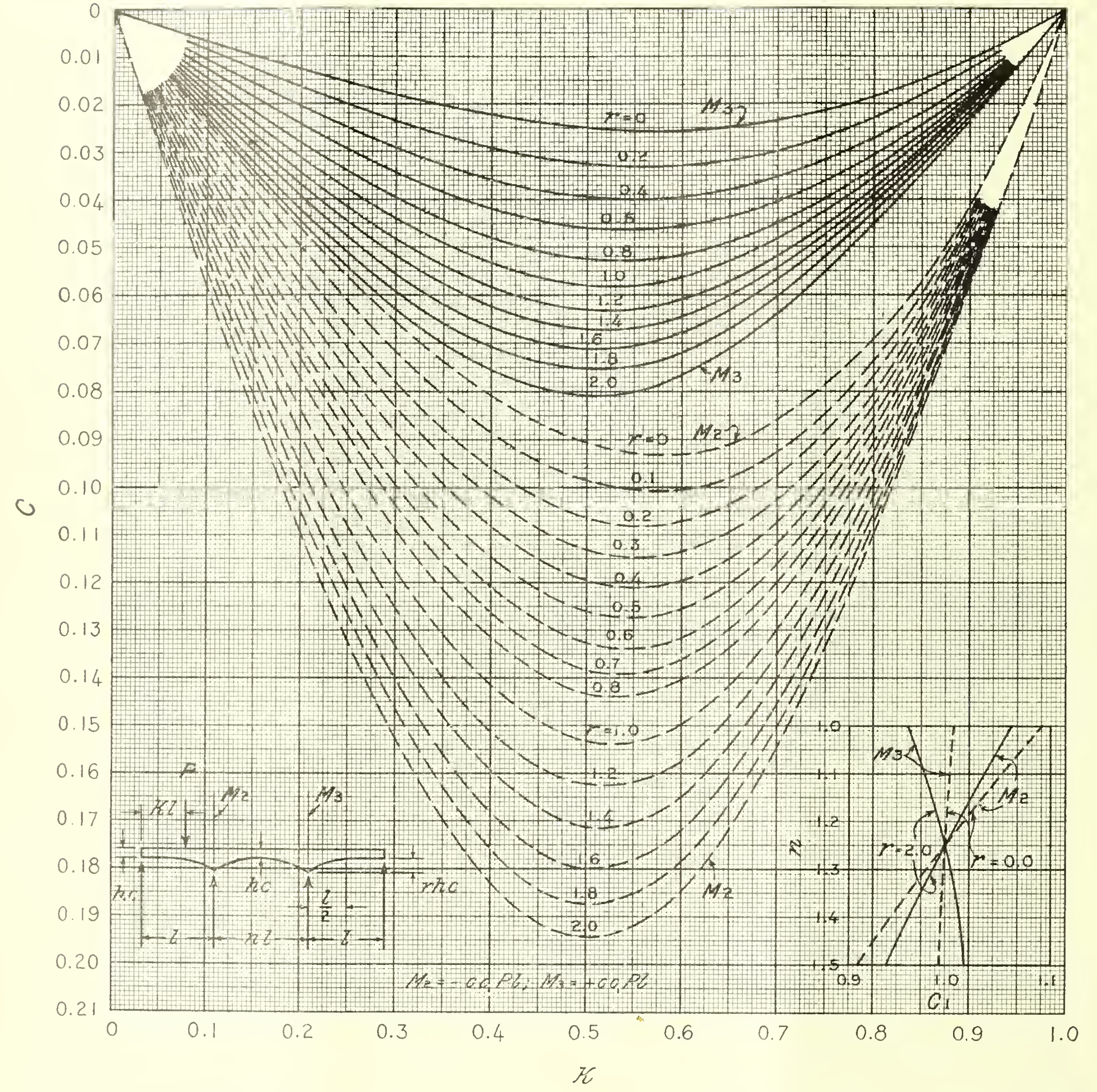


Figure 8.—Diagram for Determining Moments at Interior Supports Produced by Concentrated Loads on End Span.

$c=0.163$; $c_1=0.975$.

$M_2=-cc_1 Pl=-0.163\times0.975\times13.2\times60=-126.0$ kip-feet.

Use figure 9 and compute M_2, when $P=13.2$; $r=1.2$; $n=1.333$.

$K=0.41$; $c_1=1.09$; $c=0.148$.

$M_2=-cc_1 Pl=-0.148\times1.09\times13.2\times60=-127.7$ kip-feet.

As 127.7 is greater than 126.0, the concentrated load will be placed on span 2 as giving maximum moment at R_2.

Summary for maximum negative moment at R_2.

−1,160.0 kip-feet, dead load moment,
− 386.6 kip-feet, uniform live load moment,
− 127.7 kip-feet, concentrated live load moment,
− 138.9 kip-feet, impact (figured at 0.27),

−1,813.2 kip-feet, maximum negative moment at R_2.

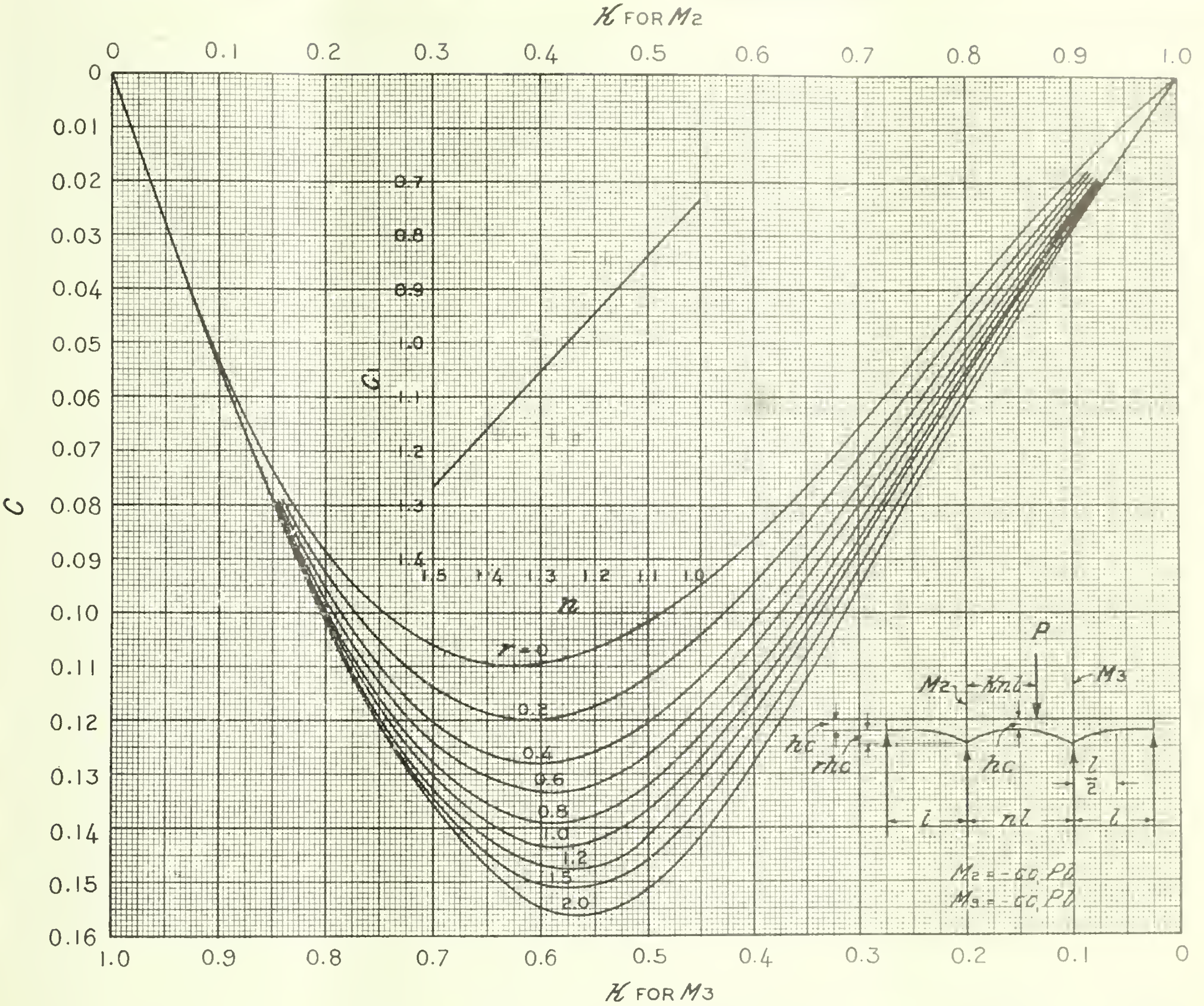


Figure 9.—Diagram for Determining Moments at the Interior Supports Produced by Concentrated Loads on Center Span

MAXIMUM POSITIVE MOMENT IN SPAN 2

The maximum positive moment in span 2 will be at the center of the span when span 2 is fully loaded with uniform live load and when the concentrated live load is placed at the center of span.

Compute the dead load moment at the center of span 2. The dead load moments at R_2 and R_3 have previously been determined as −1,160.0 kip-feet.

$.125 \times 1.543 \times 80^2 = +1{,}234.4$ kip-feet,
$13.85 \times 10 = +\ 138.5$ kip-feet,

+1,372.9 kip-feet (simple span moment for dead load),
−1,160.0 kip-feet, dead load moments at R_2 and R_3

+ 212.9 kip-feet, dead load moment at center of span 2.

Compute the live load moment at the center of span 2 with uniform live load on span 2 only and with the concentrated load at the center of span. The moments at the supports are first to be determined from the charts and then the moment at the center is computed by statics.

The moment at R_2 or R_3 for the uniform live load on span 2 as previously determined is −216.1 kip-feet.

The moment at R_2 or R_3 for the concentrated live load at the center of span 2 may be determined from figure 9.

$K = 0.5,\ n = 1.333,\ r = 1.2,\ P = 13.2,\ c = 0.141,$
$c_1 = 1.09.$
$M_2 = M_3 = -cc_1Pl$
$M_2 = M_3 = -0.141 \times 1.09 \times 13.2 \times 60 = -121.7$ kip-feet.

−216.1
−121.7

−337.8 kip-feet, live load moment at R_2 or R_3 with live load on span 2 in position for maximum positive moment in the span.

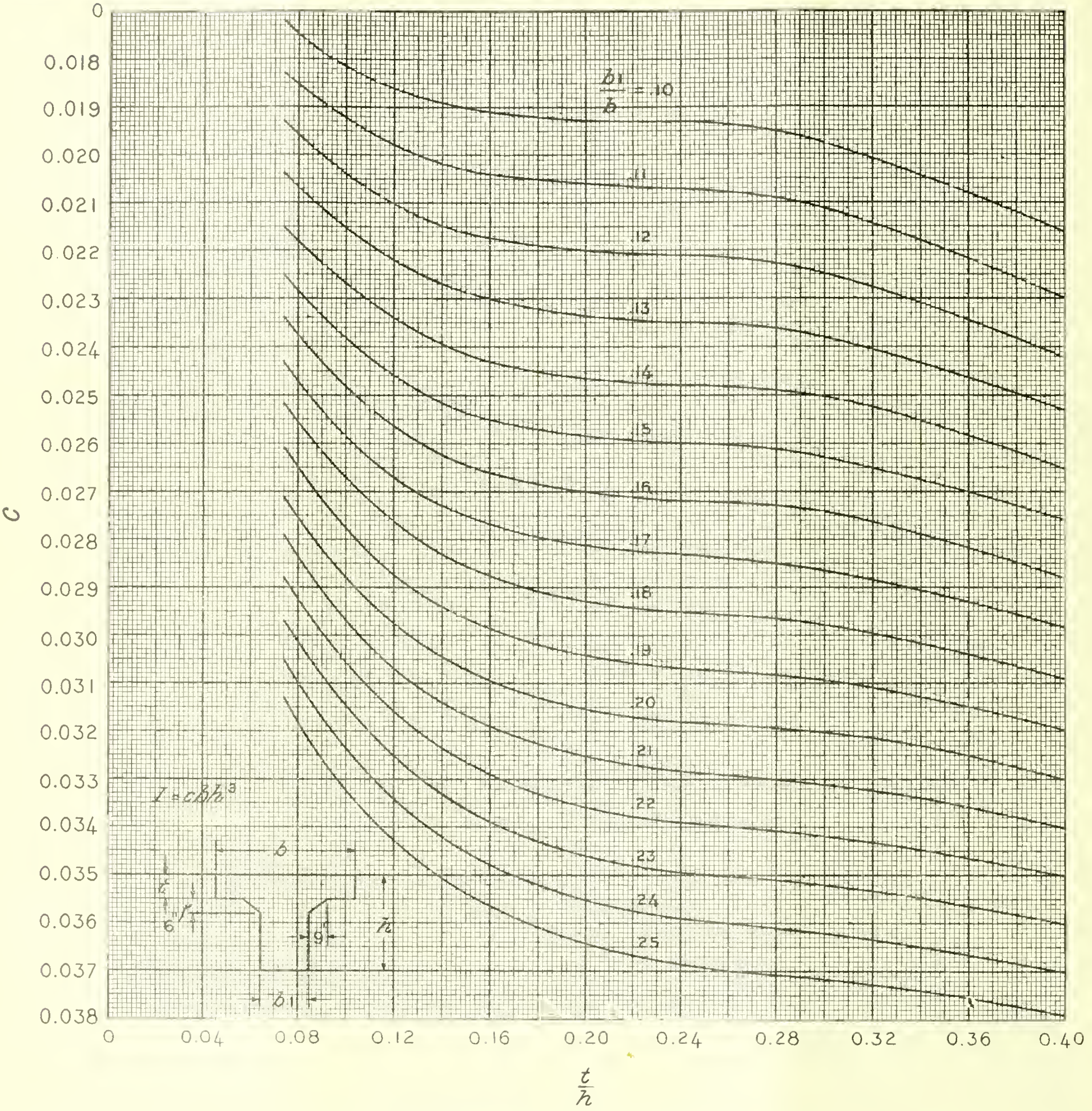


Figure 10. Moments of Inertia of T-Beams

$0.125 \times 0.469 \times 80^2 = +375.2$ kip-feet,
$0.25 \times 13.2 \times 80 = +264.0$ kip-feet,

+639.2 kip-feet (simple span moment for live load),
−337.8 kip-feet, live load moment at R_2 and R_3,

+301.4 kip-feet, live load moment at center of span 2.

Summary of maximum positive moments in span 2.

+212. 9 kip-feet, dead-load moment,
+301. 4 kip-feet, live-load moment,
+81. 4 kip-feet, impact,

+595. 7 kip-feet, maximum positive moment at center of span 2.

MAXIMUM POSITIVE MOMENT IN END SPAN

The point of maximum positive moment in the end span is not apparent from an inspection of the figures or the girder layout, and is obtained by computing the moments for the various loads at sufficient points on the span, so that these moments can be combined for the various points, and a maximum moment curve drawn. For the preliminary investigation to determine the size and shape of the girder, it will be sufficiently accurate to compute the moment at a point whose distance from the end support is $0.375\ l$.

$$0.375\times 60=22.5 \text{ feet from end support}$$

Dead-load moment.

$$22.5\times 37.5\times 0.5\times 1.543=+650.9$$

$$\frac{10.4\times 7.5\times 22.5}{60}=+29.3$$

$$0.375\times 1{,}160.0=-435.0$$

$+245.2$ kip-feet, dead load moment.

Live-load moment.—The maximum positive live-load moment at the point chosen will occur when the two end spans are loaded with uniform live load and when the concentrated live load is placed at the point.

Compute the moments at the supports for the uniform live load on the end spans.

The moment at R_2 for uniform live load on span 1 has previously been determined as -170.5 kip-feet. The moment M_2 for the uniform load on span 3 will be the same as the moment M_3 for the uniform load on span 1.

$w=0.469$, $l=60$, $r=1.2$, $n=1.333$, from figure 6, $c=\ 0.04$.

$M_3=+0.04\times 0.469\times 60^2=+67.6$ kip-feet.

$-170.5+67.6=-102.9$ kip-feet, uniform live-load moment at R_2 or R_3 with both end spans loaded.

The uniform live-load moment at a point 22.5 feet from the end of the end spans will be:

$$22.5\times 37.5\times 0.5\times 0.469=+197.8$$

$$-0.375\times 102.9=-38.6$$

$+159.2$ kip-feet uniform live-load moment at 22.5 feet from end of end span.

Compute the moment at R_2 with the concentrated load at 22.5 feet from the end of the end span. Use figure 8.

$K=0.375$, $l=60$, $r=1.2$, $n=1.333$.

$P=13.2$ kips. From figure 8, $c=0.146$; $c_1=0.975$.

$M_2=-0.146\times 0.975\times 13.2\times 60=-112.7$ kip-feet.

The concentrated live-load moment at a point 22.5 feet from the end of the end spans will then be:

$$\frac{22.5\times 37.5\times 13.2}{60}=+185.5 \text{ kip-feet,}$$

$$-0.375\times 112.7=-42.3 \text{ kip-feet,}$$

$+143.2$ kip-feet, concentrated live load moment at 22.5 feet from end of end span.

$+159.2$ kip-feet, uniform live load moment.
$+143.2$ kip-feet, concentrated live load moment,

$+302.4$ kip-feet, maximum positive live load moment at 22.5 feet from end of end span.

$0.27\times 302.4=81.6$ kip-feet, allowance for impact for maximum positive moment in end span.

Summary of maximum positive moments for a point 22.5 feet from the end bearings.

$+245.2$ kip-feet, dead load moment,
$+302.4$ kip-feet, live load moment,
$+\ 81.6$ kip-feet, allowance for impact,

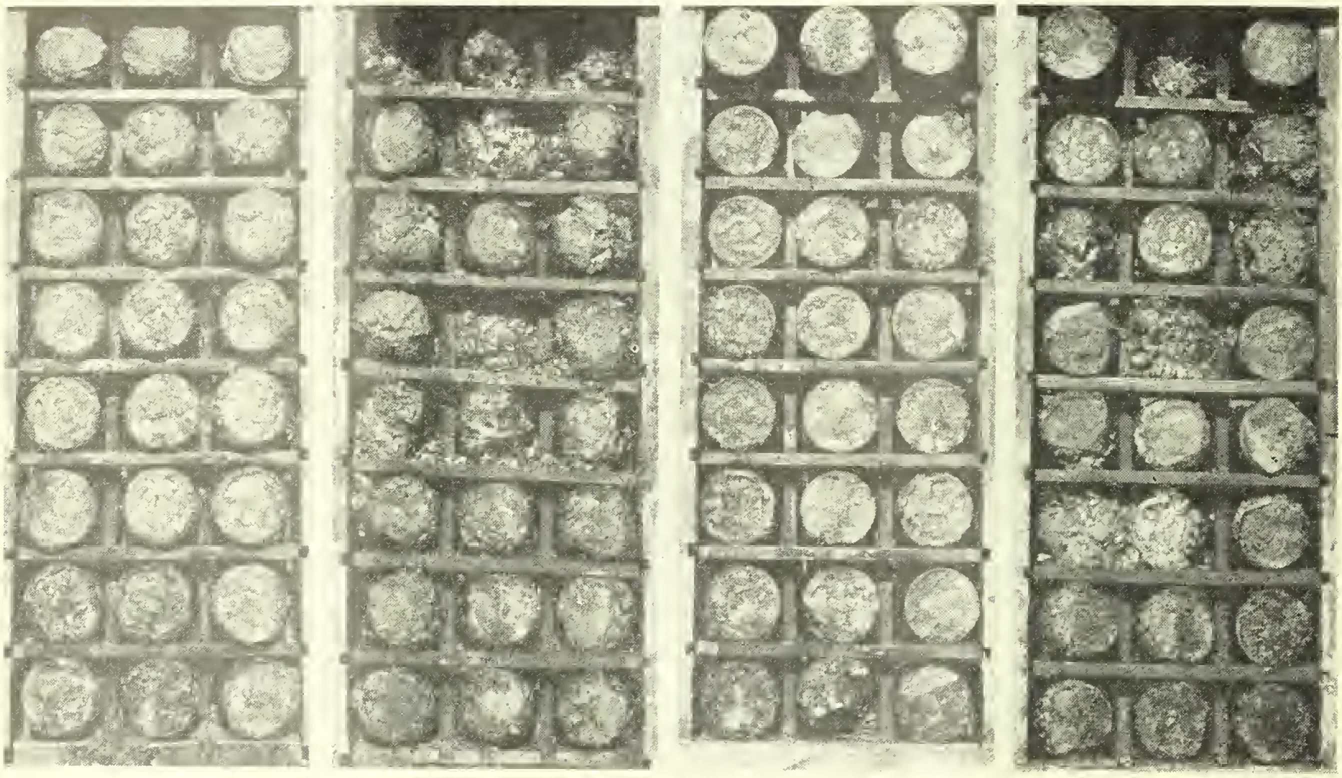
$+629.2$ kip-feet, total.

The maximum moments either positive, negative, or both, may be determined for any point, on the spans, as was done for the point 22.5 feet from the end bearings, by placing the loads on the spans in such a manner as to cause the maximum moment at the location desired.

EFFECT OF TAR, AMMONIUM FLUOSILICATE, AND SODIUM HYDROXIDE ON THE ALKALI RESISTANCE OF CONCRETE

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by E. C. E. **LORD**, formerly Associate Petrographer, and D. G. **RUNNER**, Materials Engineer



Crate No.	142	143	144	145
Cure, moist	7 days	7 days	28 days	28 days
Cure, air	1 day	7 days	1 day	7 days

Figure 1.—Condition of Concrete Specimens With Cement A, After 55 Months in Medicine Lake. W/C ratio=0.7. Two Top Rows of Each Crate Immersed ½ Hour, With Each Successive Lower Two Rows Immersed 1 Hour, 4 Hours, and 24 Hours in Water-Gas Tar. Data Given in Table 7.

EARLY INVESTIGATIONS REVIEWED

For a number of years the Public Roads Administration has been conducting alkali-resistance tests on concrete surfaces treated with various protective materials such as water-gas tar, coal tar, paraffin, lubricating oil and linseed oil. The results on the earlier tests with water-gas tar, coal tar and paraffin have been reported in Public Roads.[1] From the field and laboratory data, it was concluded that water-gas tar of proper quality is readily absorbed by cement mortar and concrete, the rate of absorption varying, however, with the manner of curing of the concrete, age, and density of the mix. Furthermore, it was concluded that four coats of water-gas tar applied at the rate of about one-fourth gallon per square yard of surface, followed by one coat of coal tar appears to afford adequate protection against alkali attack, provided the concrete is of good quality, has been properly fabricated and is not leaner than a 1 : 2 : 4 mix.

Results of subsequent investigations on alkali resistance using water-gas tar, coal tar, lubricating oil and linseed oil have also been published in Public Roads.[2] From the test data relative to concrete protection, it was found that concrete of high cement content, cured for a limited period in air and treated with water-gas tar, is capable of offering appreciable resistance to alkali attack. It was observed that the protection against alkaline waters is influenced by the cement content, the method of curing and the consistency of the mix. Rich mixes of medium consistency cured for a minimum length of time in air gave best results.

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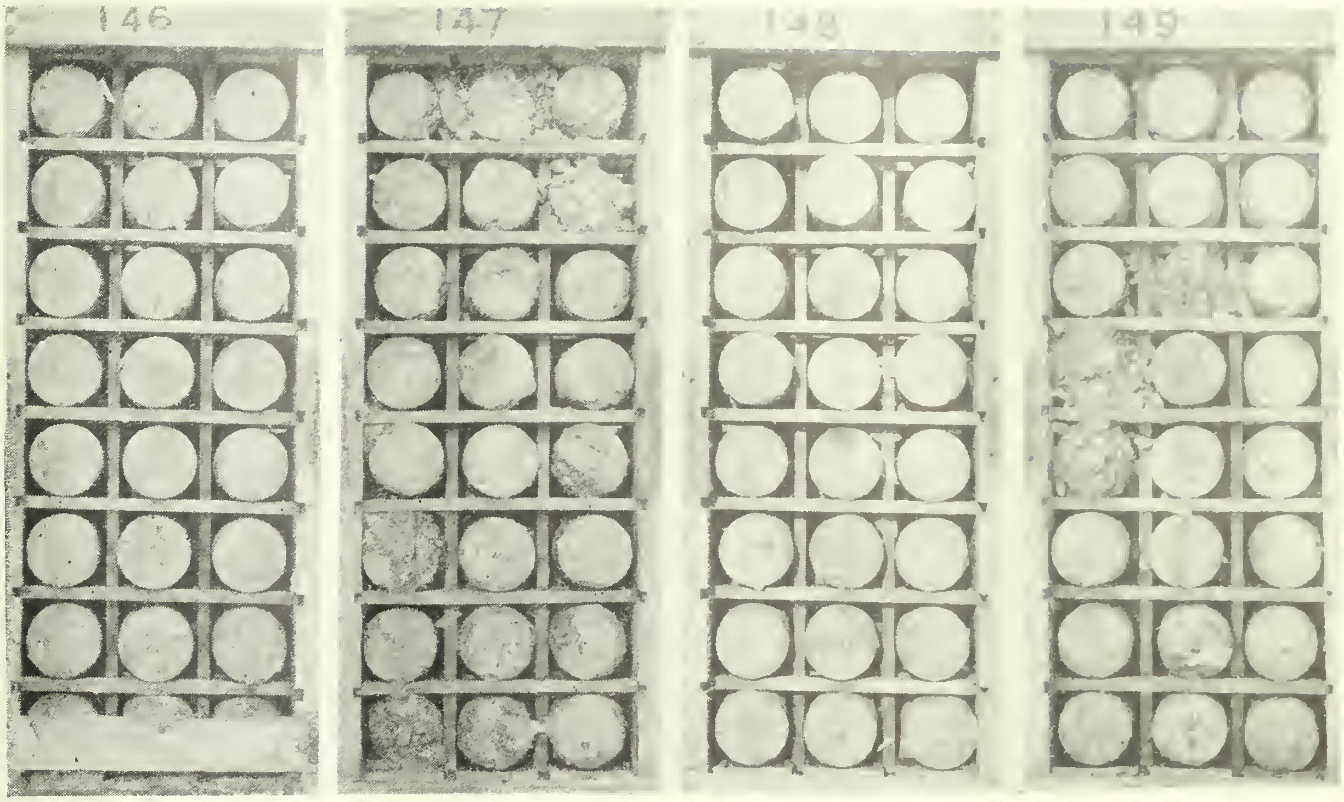
[1] The Protection of Concrete Against Alkali, vol. 5, No. 3, May 1924; vol. 6, No. 11, January 1926; vol. 8, No. 6, August 1927.

[2] The Effect of Water-Gas Tar on the Strength and Alkali Resistance of Concrete, vol. 12, No. 4, June 1931.



Crate No	146	147	148	149
Cure, moist	7 days	7 days	28 days	28 days
Cure, air	1 day	7 days	1 day	27 days

FIGURE 2.—CONDITION OF CONCRETE SPECIMENS WITH CEMENT A, AFTER 65 MONTHS IN MEDICINE LAKE. W/C RATIO=0.8. SAME PROTECTIVE TREATMENT AS GIVEN SPECIMENS IN FIGURE 1, PLUS ONE COAT OF COAL TAR. DATA GIVEN IN TABLE 8.

NEW SERIES OF TESTS DESCRIBED

In the investigations reported here, the tests were made on concrete specimens having water-cement ratios of 0.7 and 0.8 by volume. The aggregates were Potomac River sand having a fineness modulus of about 2.65 and Potomac River gravel containing 50 percent of ¾ to ½-inch size and 50 percent of ½-inch to No. 4 size.

Two brands of cement passing all the standard specification requirements were used in preparing the specimens. The results of physical and chemical tests on these cements are shown in table 1. The mix data and strength of the concrete are given in table 2. Comparing the chemical composition of the cements, it will be noted that while the major constituents (lime and silica) are about the same, the iron and alumina are present in widely different proportions. The concrete specimens made with cement B having a low alumina-iron ratio, as will be shown later, proved much more alkali resistant than those containing cement A with a high alumina-iron ratio.

The concrete specimens were 3- by 6-inch cylinders. For each water-cement ratio, curing conditions and protective treatment, a batch of three cylinders was cast for storage in sulphate solution in the laboratory and a batch of six cylinders was cast for storage in the waters of Medicine Lake, S. Dak.

For all methods of treatment the specimens were given an initial curing in the moist room of 7 days and 28 days, followed by curing in laboratory air. The periods of air curing were 1 and 7 days for the specimens to be treated with water-gas tar and 7 and 28 days for

TABLE 1.—*Properties of cement used in concrete specimens*

PHYSICAL PROPERTIES

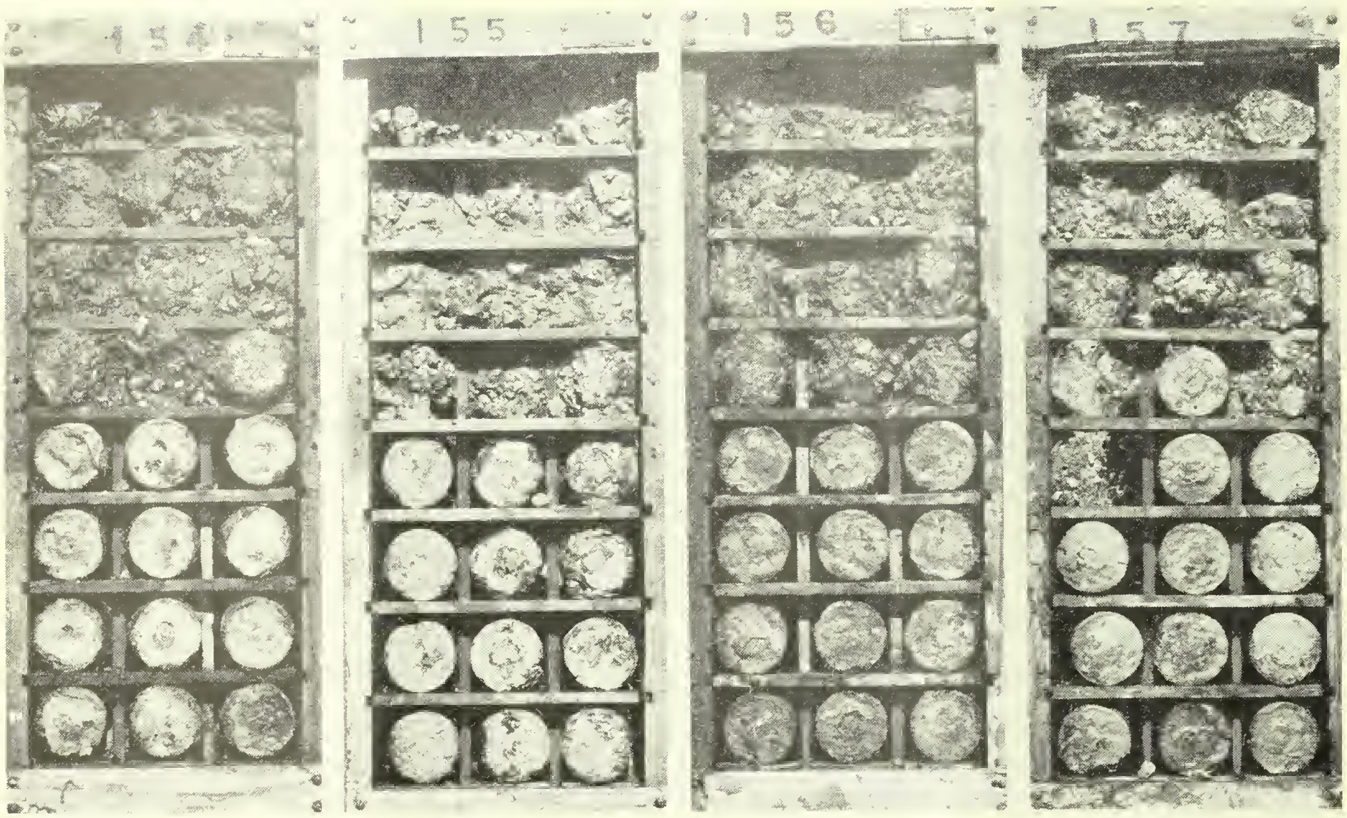
Item	Cement A	Cement B
Apparent specific gravity	3.14	--
Percentage retained on No. 200 sieve	13.9	8.5
Normal consistency, percent	23.0	23.6
Time of set, hours:		
Initial	2.3	5.5
Final	4.0	6.8
Tensile strength, lb. per sq. in.:		
7 days	325	335
28 days	385	390

CHEMICAL ANALYSES

Oxides	Percent	Percent
Silica (Si O₂)	21.25	[illegible]
Iron (Fe₂O₃)	2.84	8.04
Alumina (Al₂O₃)	6.16	4.76
Lime (CaO)	62.50	[illegible]
Magnesia (MgO)	2.96	[illegible]
Sulphuric anhydride (SO₃)	1.60	[illegible]
Loss on ignition	[illegible]	[illegible]
Ratio alumina to iron	2.17	[illegible]

COMPUTED COMPOUND COMPOSITIONS

Compounds	Percent	Percent
Tricalcium silicate (C₃S)	[illegible]	[illegible]
Dicalcium silicate (C₂S)	[illegible]	[illegible]
Tricalcium aluminate (C₃A)	12	[illegible]
Tetracalcium alumino-ferrite (C₄AF)	[illegible]	[illegible]
Dicalcium ferrite (C₂F)	[illegible]	[illegible]
Calcium sulphate (CaSO₄)	[illegible]	[illegible]



Crate No.	154	155	156	157
Cure, moist	7 days	7 days	28 days	28 days
Cure, air	1 day	7 days	1 day	7 days

FIGURE 3.—CONDITION OF CONCRETE SPECIMENS WITH CEMENT A, AFTER 74 MONTHS IN MEDICINE LAKE. W/C RATIO=0.8. TWO TOP ROWS OF EACH CRATE GIVEN 4 COATS OF WATER-GAS TAR, THIRD AND FOURTH ROWS GIVEN 6 COATS OF WATER-GAS TAR. LOWER HALF OF CRATES GIVEN SAME TREATMENT AS UPPER HALF, PLUS ONE SEAL COAT OF COAL TAR. DATA GIVEN IN TABLES 9 AND 10.

TABLE 2.—*Mix data and compressive strength of concrete specimens*

W/C ratio by volume	Mix (dry rodded volume)	Cement factor	Slump	Flow
		Sacks per cubic yard	*Inches*	*Percent*
0.8	1:1.96:3.36	5.77	3½	185
0.7	1:1.70:2.90	6.57	3	185

COMPRESSIVE STRENGTH[1]

W/C ratio	Cured		Specimen containing cement A cured in air as indicated, followed by storage in moist air for—			Specimen containing cement B cured in air as indicated, followed by storage in moist air for—		
	Moist air	Laboratory air	9 days	30 days	365 days	9 days	30 days	365 days
	Days	*Days*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*
0.8	7	7	3,820	4,490	5,050	3,590	3,870	5,320
0.8	7	28	4,020	4,570	5,940	3,700	4,350	5,730
0.7	7	7	4,120	4,700	6,100	4,770	5,550	7,070
0.7	7	28	4,540	4,930	6,400	5,480	5,500	7,050

[1] Strength tests were made on batches of 3 cylinders, each stored in moist air for the periods indicated.

the specimens to be treated with ammonium fluosilicate and sodium hydroxide.

The protective materials used in these tests were water-gas tar, ammonium fluosilicate and sodium hydroxide, applied by immersion and by brush coating, followed in some cases by a seal coat of coal tar applied with a brush.

The tars were similar to those previously used, conforming to provisional specifications TW-1-X for water-gas tar and RTCB-5 for coal tar. The detail requirements for these materials are given in table 3.

TABLE 3.—*Requirements for water-gas tar and coal tar*

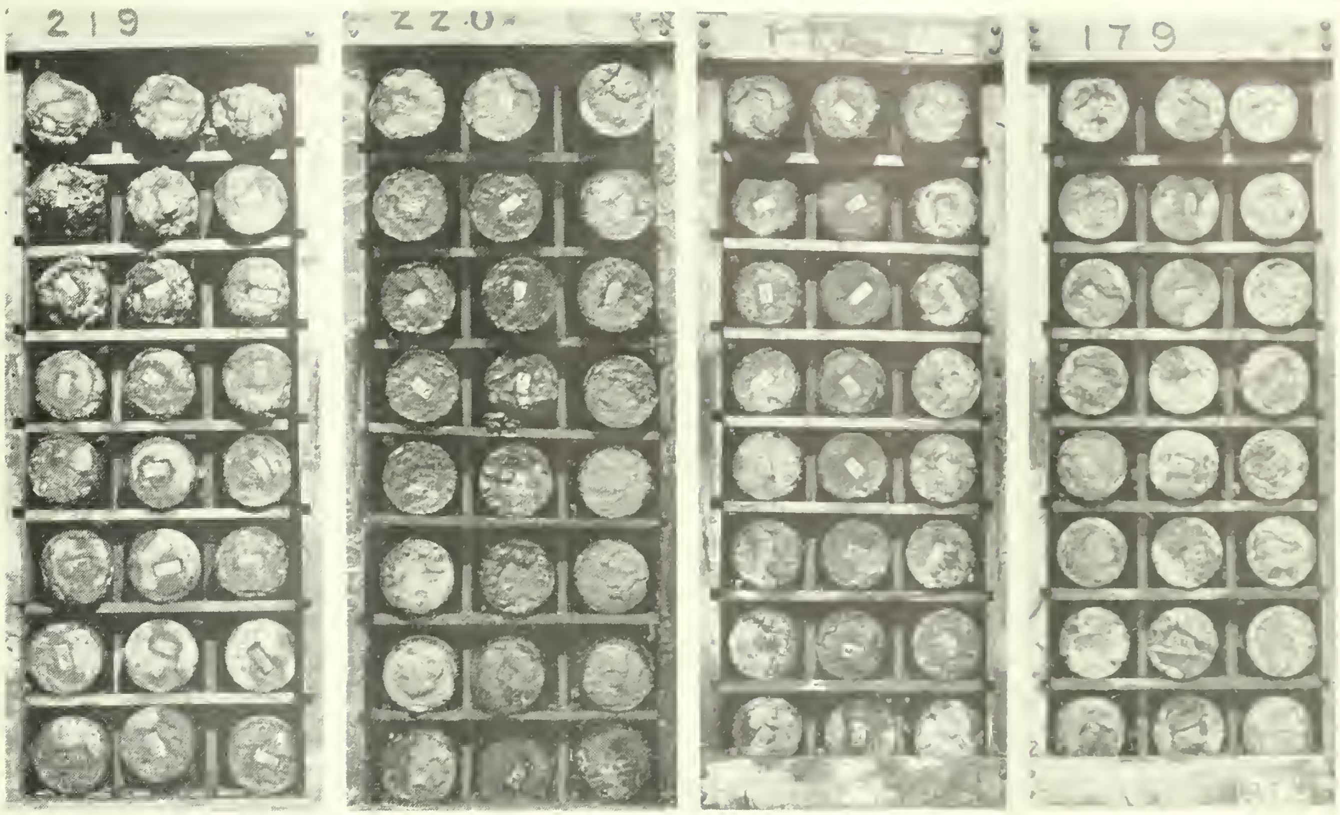
Grade	TW-1-X[1]	RTCB-5[2]
1. Specific gravity, 25°/25° C. (77°/77° F.)	1.030-1.100	1.09+
2. *a.* Specific viscosity, 40° C	3.0−	
b. Specific viscosity, 50° C		17-26
3. Total distillate, percentage by weight:		
To 170° C		2.0-8.0
To 200° C		5.0+
To 235° C		8.0-18.0
To 300° C	50.0−	35.0−
a. Softening point of residue, ° C		40-70
4. Total bitumen, percentage by weight	98+	80+
5. Water, percentage by volume	3.0−	1.0−

[1] Water-gas tar for absorptive treatment shall be crude water-gas tar, which may be treated for removal of excess water if necessary to meet the detail requirements of the specification.

[2] Fluxed refined tar for seal application shall be prepared from refined gas house, coke-oven, and/or water-gas tars fluxed with suitable distillates.

Ammonium fluosilicate and sodium hydroxide were applied in solutions of 20 and 10 percent, respectively, by weight of the air-dried compounds.

As previously indicated, part of the specimens were stored in a sulphate solution in the laboratory and part of them were stored in the sulphate waters of Medicine Lake, S. Dak. The laboratory storage was in a solution containing 5 percent sodium sulphate and 5 percent magnesium sulphate. The waters of Medicine Lake



Crate No.	219	220	178	179
Cement	A	A	B	B
Cure, moist	7 days	7 days	7 days	7 days
Cure, air	7 days	28 days	7 days	28 days
Months in lake	37	37	62	61

Figure 4.—Condition of Concrete Specimens After Indicated Number of Months in Lake. W/C Ratio = 0.8. Two Top Rows of Each Crate Immersed ½ Hour, With Each Successive Lower Two Rows Immersed 1 Hour, 4 Hours, and 24 Hours in Ammonium Fluosilicate Plus Equivalent Periods of Immersion in Water-Gas Tar. Data Given in Tables 11 and 12.

contain high percentages of salts. Analyses made in 1923, 1924, and 1925 indicated a total salt content of from 2.34 to 4.72 percent, consisting almost entirely of a combination of magnesium and sodium sulphates in which the magnesium salt predominated.[3] A sample of water taken from the lake in July 1941 showed the following composition:

Magnesium sulphate	3.3 percent.
Sodium sulphate	1.4 percent.
Calcium sulphate	.3 percent.
Sodium chloride	.1 percent.
Calcium carbonate	.06 percent.
Specific gravity	1.0532

METHODS OF TREATMENT DESCRIBED

The protective treatments were applied to 16 major groups of specimens, each group containing 8 subgroups of either 2 or 4 batches of cylinders each. Batches for the laboratory storage contained 3 cylinders each and those for lake storage contained 6 cylinders each. The general schedule for the entire series is indicated in table 4. The major groups of specimens numbered 1 to 10, inclusive, were treated with water-gas tar and ammonium fluosilicate, and stored both in the laboratory and in Medicine Lake. The major groups numbered 5 S to 10-S, inclusive, constituted a supplementary series in which the primary treatment was with sodium hydroxide, the specimens being stored in the laboratory only.

Cement A was used throughout the entire investigation, while cement B was used only in the ammonium fluosilicate series.

In the earlier investigations it was found that the curing period was a factor in the absorption of the protective agents. In order to further investigate this effect, the concrete specimens were subjected to different periods of moist curing and air curing. After air curing, the protective treatments were applied.

The materials used in the initial treatments were water-gas tar, ammonium fluosilicate, and sodium hydroxide. These were applied both by immersion and by brush coating. For some specimens this was the final treatment. For other specimens the initial treatment was followed by a seal of coal tar consisting of one coat applied with a brush. Other specimens were given an initial treatment with ammonium fluosilicate or sodium hydroxide followed by an immersion or brush treatment with water-gas tar.

Immediately after air curing, the initial weights of the specimens were determined and they were then given the protective treatment. They were then weighed a second time to determine the absorption or retention of the protective liquid. After a period of drying and immediately before immersion in the sulphate solution, they were weighed again. This third weight was used in determining the change in weight that took place in the solution.

The specimens to be treated by immersion were weighed and immersed in the protective liquid for the

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[3] Public Roads, vol. 6, No. 8, October 1925, p. 174.

TABLE 4.—*Schedule of protective treatments*

Major group No.	Cement	Water-cement ratio	Curing		Protective material	Method of application	Period of immersion (hours)	Number of brush coats
			Moist	Air				
			Days	*Days*				
1	A	0.7 and 0.8	7 and 28	1 and 7	Water-gas tar	Immersion	½, 1, 4, and 24	
2	A	do	do	do	Water-gas tar followed by seal coat of coal tar	do Brush coat	½, 1, 4, and 24	1.
3	A	do	do	do	Water-gas tar	do		4 and 6.
4	A	do	do	do	Water-gas tar followed by seal coat of coal tar	do do		4 and 6. 1.
5	A and B	do	do	7 and 28	Ammonium fluosilicate	Immersion	½, 1, 4, and 24	
6	A and B	do	do	do	Ammonium fluosilicate followed by seal coat of coal tar	do Brush coat	½, 1, 4, and 24	1.
7	A and B	do	do	do	Ammonium fluosilicate followed by water-gas tar[1]	Immersion[1]	½, 1, 4, and 24	
8	A and B	do	do	do	Ammonium fluosilicate	Brush coat		4 and 6.
9	A and B	do	do	do	Ammonium fluosilicate followed by seal coat of coal tar	do do		4 and 6. 1.
10	A	do	do	do	Ammonium fluosilicate followed by water-gas tar	do do		4 and 6. 3.
10	B	do	do	do	Ammonium fluosilicate followed by water-gas tar	do do		4 and 6. 2.
5-S	A	do	do	do	Sodium hydroxide	Immersion	½, 1, 4, and 24	
6-S	A	do	do	do	Sodium hydroxide followed by seal coat of coal tar	do Brush coat	½, 1, 4, and 24	1.
7-S	A	do	do	do	Sodium hydroxide followed by water-gas tar	Immersion[2]	½, 1, 4, and 24	
8-S	A	do	do	do	Sodium hydroxide	Brush coat		4 and 6.
9-S	A	do	do	do	Sodium hydroxide followed by seal coat of coal tar	do do		4 and 6. 1.
10-S	A	do	do	do	Sodium hydroxide followed by water-gas tar	do do		4 and 6. 4.

[1] The periods of immersion in water-gas tar and ammonium fluosilicate were the same. That is, specimens immersed for ½ hour, 1 hour, etc., in fluosilicate were then immersed for ½ hour, 1 hour, etc., respectively, in water-gas tar.

[2] The immersion procedure was the same as described in footnote 1 except that sodium hydroxide was used instead of ammonium fluosilicate.

TABLE 5.—*Effect of sulphate solution on untreated concrete stored in laboratory*[1]

CEMENT A

Series No.	Batch No.	W/C ratio	Cured—days		Percentage of gain in weight after laboratory storage																			
			Moist	Air	¼ month	5 months	7 months	8 months	9 months	10 months	12 months	20 months	21 months	31 months	32 months	33 months	43 months	44 months	45 months	68 months	69 months	91 to 93 months[2]	102 months	120 months
1	1	0.8	7	1	1.94				3.27	F														
	2		7	7	2.55					3.56	F													
	3		28	1	1.57				3.77	F														
	4		28	7	2.13				3.82	F														
2	5	.7	7	1	1.44					2.59	F													
	6		7	7	2.10					3.00	F													
	7		28	1	1.11				1.92	F														
	8		28	7	1.80				2.45	F														
3	9	.8	7	7	2.68	3.97		5.35	F															
	10		7	28	3.72	4.34		5.88	F															
	11		28	7	1.75	1.88		3.70	F															
	12		28	28	2.28	2.51		5.29	F															
4	13	.7	7	7	2.16	2.45		2.64			2.96	F												
	14		7	28	3.10	3.93		4.08			5.32	F												
	15		28	7	1.66	1.88		1.95			2.86	F												
	16		28	28	2.79	2.98		2.96			3.46	F												

CEMENT B

Series No.	Batch No.	W/C ratio	Moist	Air	¼ month	5 months	7 months	8 months	9 months	10 months	12 months	20 months	21 months	31 months	32 months	33 months	43 months	44 months	45 months	68 months	69 months	91 to 93 months[2]	102 months	120 months
5	17	0.8	7	7	2.35				2.81				3.04			3.10			3.55		4.19	F (92)		
	18		7	28	3.48			3.76				4.19			4.36				5.04		5.80	F (92)		
	19		28	7	1.97				2.35			2.60			2.77				3.03		3.41	F (92)		
	20		28	28	2.85			3.36				3.55			3.76		4.57			4.95		F (92)		
6	21	.7	7	7	2.16				2.66				2.81			2.93			3.06		3.60	F (93)		
	22		7	28	3.08			3.33				3.33				3.64		3.73		5.34		F (92)		
	23		28	7	1.40		1.80					1.96		2.08			2.37			2.60		3.05 (91)	3.26	3.26
	24		28	28	1.66		2.28					2.51		2.63			2.90			3.27		3.92 (91)	4.04	4.04

[1] Specimens were tested in 6 series, each containing 4 batches of 3 cylinders to a batch. F indicates failure of specimens.

[2] Figures in parentheses indicate exact number of months in solution at time of weighing or number of months in solution until failure.

desired period of time. The periods were ½, 1, 4, and 24 hours. At the end of the immersion period the specimens were removed from the treating bath and allowed to drain for 1 hour before the second weight determination was made. If the specimens were to receive only this initial treatment, they were allowed to dry for 2 days, weighed for the third time, and immersed in the sulphate solution. In the case of groups 7 and 7–S, the initial immersion in ammonium fluosilicate or sodium hydroxide, was followed by immersion for equal periods of time in water-gas tar. After the first immersion, the specimens were allowed to dry before being immersed in the water-gas tar but otherwise the procedure was the same as for a single immersion.

The specimens for brush treatment were weighed and given 4 and 6 brush coats of the protective liquid, each coat being permitted to dry before the next coat was applied. After the final drying they were weighed and then, if no further treatment was intended, they were dried for an additional period of 2 days, weighed again, and immersed in the sulphate solution. The same procedure was followed with the specimens of groups 10 and 10-S except that the initial brush treatments with ammonium fluosilicate and sodium hydroxide, respectively, were followed by brush treatments with water-gas tar.

The initial treatment of the specimens in groups 2, 4, 6, 9, 6–S, and 9–S was followed by a seal treatment of 1 brush coat of coal tar. These specimens were allowed to dry for 2 days after the initial treatment, the brush coat of coal tar was applied, and they were dried for an additional period of 2 days before being exposed to the alkali solution in the laboratory.

The specimens intended for storage in Medicine Lake were given the same treatments as those intended for laboratory storage. They were shipped in specially prepared crates containing 24 cylinders each, and were immediately placed in the lake, where they were inspected and photographed at yearly intervals at which time the condition of the specimens was recorded.

The laboratory specimens were placed in covered, porcelain-lined 5-gallon cans containing the sulphate solution.[4] After 7 days they were removed from the sulphate solution, dried for ½ hour in air and the gain in weight, based upon the dry weight after treatment, recorded. They were then transferred to paraffin-coated concrete storage tanks containing the sulphate solution and the gain in weight determined as before at intervals thereafter, or until failure.

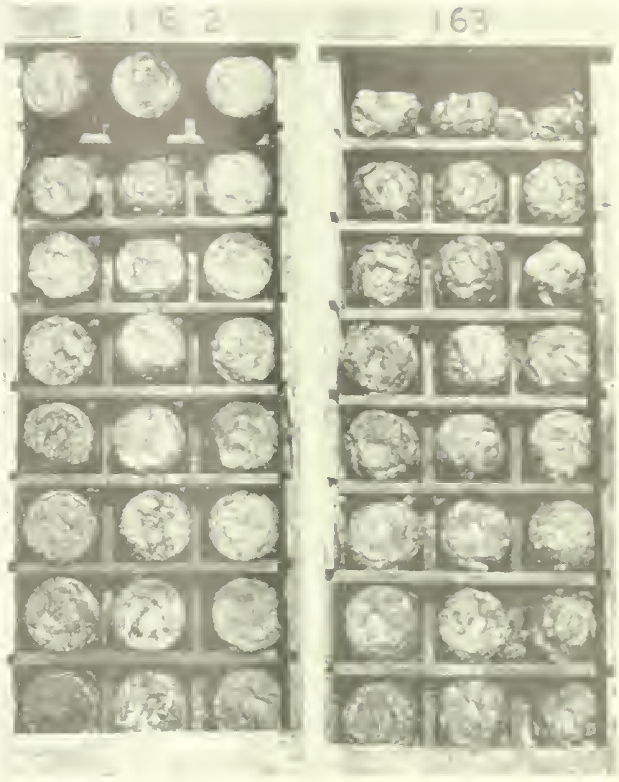
In addition to the treated specimens that have been described, batches of cylinders without treatment were made for each brand of cement, water-cement ratio and method of curing for exposure to the sulphate solution. Immediately after the period of air curing they were immersed in the sulphate solution and weighed at intervals until failure. These specimens were allowed to remain in the original containers and were not transferred to the storage tanks.

In all these laboratory tests, no attempt was made to maintain the initial concentration of the sulphate solution. The original volume of solution was maintained constant during the period of the tests by the addition of distilled water but no sulphates were added.

EFFECT OF ALKALI ON UNTREATED CONCRETE INVESTIGATED

Before discussing the protective effect of the compounds used in this investigation, it will be of interest to note the relative resistance to alkali attack offered by untreated concrete made with cement brands A and B. The results of these exposure tests are given in table 5.

It will be noted that three series containing cement A failed at 12 months or less, while one series failed at 20 months. On the other hand, all series containing cement B were intact at the end of 69 months. It appears that, for the curing conditions studied, the resistance to alkali attack, as measured by gain in weight, was increased as the period of moist curing was increased and as the period of air curing was decreased. This condition was noted for concrete of both 0.7 and 0.8 water-cement ratio. For comparable curing conditions the concrete having the lower water-cement ratio was the more resistant.



Crate No.	162	163
Cure, moist	7 days	7 days
Cure, air	7 days	28 days
Months in lake	43	42

FIGURE 5.—CONDITION OF CONCRETE SPECIMENS WITH CEMENT B, AFTER NUMBER OF INDICATED MONTHS IN MEDICINE LAKE. W/C RATIO—0.8. TWO TOP ROWS OF EACH CRATE IMMERSED ½ HOUR, WITH EACH SUCCESSIVE LOWER TWO ROWS IMMERSED 1 HOUR, 4 HOURS, AND 24 HOURS IN AMMONIUM FLUOSILICATE. DATA GIVEN IN TABLE 11.

PROTECTIVE VALUE OF WATER-GAS TAR, APPLIED BY IMMERSION, STUDIED

Exposure tests were made on concrete specimens containing low-alkali-resistant cement A treated with water-gas tar by immersion and by brush coats and both with and without seal coats of coal tar. The results of these tests on specimens stored in the laboratory and in Medicine Lake are given in tables 6, 7, 8, and 9, corresponding to major groups 1, 2, 3, and 4. The results of water-gas tar treatment by immersion alone are given in table 6.

It will be noted from the table that the average quantity of tar absorbed increased with the time of immersion, was greater in the concrete of 0.8 water-cement ratio than in that of 0.7 water-cement ratio, and was correspondingly greater in the specimens cured for 7 days in air than in those cured in air for 1 day. The average percentage of tar absorbed by samples having the same air curing was uniformly greater for the specimens moist cured for 7 days than for those moist cured for 28 days.

In regard to the protective effect of the tar treatment, the laboratory results indicate that the leaner mix (higher water-cement ratio), although more tar absorbent, was somewhat less sulphate resistant than the

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[4] This solution was made up in 5-gallon lots, each containing 2.25 pounds $MgSO_4 \cdot 7H_2O$ and 1 pound anhydrous Na_2SO_4.

TABLE 6.—*Protective effect of water-gas tar applied by immersion on concrete. Specimens stored in sulphate solution, major group 1, cement A* [1]

Subgroup No.	Crate No.	W/C ratio	Cured—days		Treatment, hours in water-gas tar	Percentage of water-gas tar absorbed	Percentage of gain in weight after laboratory storage				Number of lake samples attacked and failed after—					
			Moist	Air			¼ month	9 or 10 months	20 to 22 months [2]	22 to 33 months [2]	8 months	19 months	31 months	43 months	55 months	67 months
1	138	0.8	7	1	½	0.63	0.99	1.73	4.74 (21)	F (23)	0	0	6	6	F	
					1	.77	.91	1.40	4.25 (21)	F (23)	0	0	6	6	F	
					4	1.03	.57	1.01	4.28 (21)	F (33)	0	0	0	6	F	
					24	1.37	.44	.80	2.57 (21)	F (33)	0	0	0	6	F	
Average						.95	.73	1.23	3.96							
2	139	.8	7	7	½	1.03	1.37	2.29	F (21)		0	6	6	F		
					1	1.40	1.28	2.15	F (21)		0	6	6	F		
					4	1.82	.90	1.93	F (21)		0	0	6	F		
					24	2.34	.66	1.38	3.09 (21)	F (33)	0	0	6	6	F	
Average						1.65	1.05	1.94								
3	140	.8	28	1	½	.41	.94	1.75	F (22)		0	0	0	6	F	
					1	.51	.88	1.68	F (22)		0	0	0	6	F	
					4	.64	.60	1.09	4.74 (20)	F (27)	0	0	0	6	F	
					24	.82	.50	.81	2.93 (20)	F (27)	0	0	0	6	F	
Average						.60	.73	1.33								
4	141	.8	28	7	½	.67	1.45	2.42	9.23 (20)	F (22)	0	6	6	F		
					1	.82	1.18	1.94	6.33 (20)	F (22)	0	6	6	F		
					4	1.20	.90	1.49	3.90 (20)	F (27)	0	0	6	F		
					24	1.73	.58	1.10	2.38 (20)	F (27)	0	0	6	F		
Average						1.10	1.03	1.74	5.46							
5	142	.7	7	1	½	.64	.87	1.28	3.16 (21)	F (28)	0	6	6	6	6	F
					1	.73	.64	1.04	3.30 (21)	F (28)	0	0	6	6	6	F
					4	.97	.56	.90	2.73 (21)	F (31)	0	0	6	6	6	F
					24	1.15	.50	.75	2.25 (21)	F (31)	0	0	0	3	6	F
Average						.87	.64	.99	2.86							
6	143	.7	7	7	½	.63	1.22	1.70	5.46 (21)	F (22)	0	6	6	6	F	
					1	.90	1.02	1.48	4.73 (21)	F (22)	0	6	6	6	F	
					4	1.44	.84	1.18	3.82 (21)	F (27)	0	6	6	6	F	
					24	1.82	.64	.97	2.72 (21)	F (27)	0	0	6	6	F	
Average						1.20	.93	1.33	4.19							
7	144	.7	28	1	½	.33	.64	1.00	2.63 (20)	F (27)	0	0	0	6	6	F
					1	.41	.45	.89	2.08 (20)	F (27)	0	0	0	6	6	F
					4	.56	.43	.80	1.61 (20)	F (30)	0	0	0	6	6	F
					24	.66	.27	.58	1.52 (20)	F (30)	0	0	0	6	6	F
Average						.49	.45	.82	1.96							
8	145	.7	28	7	½	.59	1.08	1.64	1.98 (20)	F (31)	0	6	6	6	F	
					1	.68	.95	1.46	2.27 (20)	F (31)	0	6	6	6	F	
					4	1.08	.69	1.22	1.47 (20)	F (31)	0	6	6	6	F	
					24	1.51	.49	1.04	1.12 (20)	F (31)	0	0	6	6	F	
Average						.96	.80	1.34	1.71							

[1] Values based on 3 and 6 specimens for laboratory and lake storage, respectively. F indicates failure of specimens.
[2] Figures in parentheses indicate exact number of months in solution at time of weighing or number of months in solution until failure.

richer mix. It will be observed too that the quantity of sulphate absorbed by each subgroup of specimens, as indicated by the gain in weight, decreased as the time of immersion in the water-gas tar was increased. The greater resistance thus indicated was reflected to some extent in the ages at which failure took place, but the differences in age at failure are not sufficiently great to be considered of practical importance. After 33 months exposure to the sulphate solution all specimens stored in the laboratory had failed.

The effect of the water-gas tar treatment on the durability of samples stored in Medicine Lake showed approximately the same trend as that indicated by the laboratory specimens. However, all the lake specimens had failed at 67 months. Comparable subgroups of the richer mix were somewhat more alkali resistant than those of the leaner mix and the beneficial effect of the shorter air curing was also indicated. (See photographs in fig. 1, crates 142 to 145 inclusive.)

EFFECT OF WATER-GAS TAR WITH COAL TAR SEAL COAT

The results of the tests on concrete treated with water-gas tar by immersion, followed by a seal coat of coal tar, are given in table 7. A comparison of these results with those given in table 6 shows that the addition of the seal coat appreciably increased the resistance of the concrete to the action of sulphate waters.

Of the specimens shown in table 6, all had failed in the laboratory at 33 months. In contrast to this, table 7 shows that none of the seal-coated specimens failed in the laboratory until about 53 months and two of the subgroups were showing continued resistance at ages of 101 and 102 months.

The absorption of tar by the specimens shown in table 7 was affected by the different variables in the same manner as the specimens of table 6. Table 7 shows that the resistance of comparable subgroups of laboratory specimens was generally greater for the water-cement ratio of 0.7 (rich mix) than for the water-cement ratio of 0.8 (lean mix); and was greater for the specimens cured 1 day in air than for those cured 7 days in air.

With minor exceptions, the same factors that influenced the resistance of the laboratory specimens also influenced the resistance of the specimens stored in Medicine Lake. Insofar as the data for both laboratory and Lake specimens permit of a determination, it appears that little, if any, added durability is gained

TABLE 7.—*Protective effect of water-gas tar applied to concrete by immersion followed by a seal coat of coal tar. Specimens stored in sulphate solution, major group 2, cement A* ¹

Subgroup No.	Crate No.	W/C ratio	Cured—days		Hours in water-gas tar	Percentage of tar absorbed		Percentage of gain in weight after laboratory storage ¹								Number of lake samples at[illegible]								
			Moist	Air		Water-gas tar	Coal tar	¼ month	7 or 8 months	41 or 42 months	53 or 54 months	65 or 66 months	77 or 78 months	101 or 102 months	113 months	29 months	41 months	53 months	65 months	77 months	89 months	101 months	113 months	125 months
9	146	0.8	7	1	½	0.60	0.56	0.35	0.64	1.10	1.23	1.43	1.95	F		0	0	0	0	6	6	[illegible]	1F	1F
					1	.71	.62	.35	.60	1.10	1.29	1.54	2.01	F		0	0	0	0	0	0			1F
					4	.86	.62	.19	.46	.85	1.04	1.37	4.47	F		0	0	0	0	6	6	2F	F	4F
					24	1.26	.62	.19	.40	.94	F					0	0	0	0	6	6	[illegible]	2F	2F
Average						.86	.61	.27	.53	1.00														
10	147	.8	7	7	½	.94	.41	.98	1.72	3.32	F					3	3	6	F					
					1	1.22	.44	.76	1.39	2.94	F					3	3	6	F					
					4	1.80	.49	.59	1.18	2.19	F					0	3	6	F					
					24	2.32	.39	.48	1.00	2.00	F					0	3	6	F					
Average						1.57	.43	.70	1.32	2.61														
11	148	.8	28	1	½	.46	.60	.48	.82	1.69	1.78	1.78	F			0	0	0	0	6	F			
					1	.59	.56	.48	.86	1.28	1.32	1.38	2.21	F		0	0	0	1	F				
					4	.75	.58	.37	.71	1.06	1.24	1.54	3.97	F		0	0	0	1	F				
					24	.85	.33	.29	.70	1.07	1.07	F				0	0	0	1	6	F			
Average						.66	.52	.40	.77	1.28	1.35													
12	149	.8	28	7	½	.53	.50	.54	1.42	3.26	F					0	0	3	6	6	F			
					1	.82	.48	.59	1.40	2.17	F					0	0	3	6	2F	F			
					4	1.19	.39	.21	.96	1.78	F					0	0	3	6	1F	3F	4F	F	
					24	1.83	.38	.04	.85	1.60	F					0	0	0	1	6	6	2F	F	
Average						1.09	.44	.35	1.16	2.20														
13	150	.7	7	1	½	.61	.44	.12	.71	1.21	1.42	1.67	2.79	F		0	0	1	2	6	6	6	6	6
					1	.61	.38	.02	.69	1.15	1.29	1.45	2.15	F		0	0	0	0	0	2	2	2	2
					4	.84	.40	0	.60	1.47	1.64	2.28	F			0	0	1	1	1	2	2	1F	2F
					24	1.04	.31	0	.56	1.12	1.16	1.41	3.22	F		0	0	1	2	6	6	6	1F	2F
Average						.77	.38	.04	.64	1.24	1.38	1.70												
14	151	.7	7	7	½	.68	.42	.50	1.20	1.75	1.88	1.92	2.13	F		0	0	0	1	6	6	6	4F	5F
					1	.95	.40	.38	1.11	1.57	1.76	1.76	2.10	2.81	F	0	0	0	0	1	1	2	4	1F
					4	1.34	.44	.27	.86	1.26	1.26	1.26	1.42	2.19	F	0	0	0	1	6	6	1F	1F	3F
					24	1.80	.44	.14	.76	1.14	1.14	1.14	2.96	F		0	0	0	1	6	6	3F	3F	4F
Average						1.19	.42	.32	.98	1.43	1.51	1.52	2.15											
15	152	.7	28	1	½	.39	.37	.23	.83	1.10	1.10	1.20	1.72	2.15	F	0	0	0	0	6	6	4F	F	
					1	.45	.39	.23	.84	.99	1.11	1.11	1.36	2.29	F	0	0	0	0	6	6	4F	F	
					4	.62	.41	.10	.64	.86	.94	.97	1.23	3.56	F	0	0	6	6	6	6	4F	F	
					24	.64	.43	.04	.60	.88	.94	.99	1.61	1.63	F	0	0	6	6	6	6	1F	F	
Average						.53	.40	.15	.73	.96	1.02	1.07	1.48	2.41										
16	153	.7	28	7	½	.40	.33	.44	1.42	2.15	F					0	0	0	0	6	6	1F	F	
					1	.57	.38	.27	1.26	2.61	F					0	0	0	1	6	6	1F	F	
					4	.91	.29	.15	1.07	1.80	F					0	0	0	1	6	6	2F	F	
					24	1.45	.28	.06	.97	2.11	F					0	0	3	2	F				
Average						.83	.32	.23	1.18	2.17														

¹ Values based on 3 and 6 specimens for laboratory and lake storage respectively. F indicates failure of specimens.

by increasing the period of immersion in water-gas tar beyond 1 hour. Photographs of crates 146 to 149, inclusive, are shown in figure 2.

PROTECTIVE VALUE OF WATER-GAS TAR APPLIED BY BRUSH COATING

The test results for concrete treated with water-gas tar applied by brush coating and for a comparable series of brush-coated specimens with a seal coat of coal tar are given in tables 8 and 9. The significant showing of these tables is the low protection afforded concrete by brush coats of water-gas tar (table 8) as compared to the protection given by the same treatment with an additional seal coat of coal tar (table 9). Over half of the entire lot of laboratory specimens with a brush coat only had failed at 40 months, while practically all the laboratory specimens with an additional seal coat were intact at 76 months. The test results indicate about the same general relations between water-cement ratio, method of curing and tar absorption as were recorded in tables 6 and 7 for the immersion treatment.

Comparing the effect of the treatments on specimens stored in Medicine Lake, it will be noted that nearly all cylinders treated with water-gas tar alone had failed after an exposure of 74 months or less (table 8), while of those receiving the coal tar seal coat, only two failures were recorded at 74 months (table 9). Photographs of crates 154 to 157 inclusive in figure 3 show the condition of four representative subgroups with and without seal coats of coal tar.

EFFECT OF AMMONIUM FLUOSILICATE ALONE AND IN COMBINATION WITH WATER-GAS TAR AND SEAL COATS OF COAL TAR

A preliminary investigation of various fluosilicate compounds indicated that the ammonium salt, when applied to concrete in a 20-percent water solution, was effective in retarding sulphate attack, especially when followed by an application of water-gas tar or coal tar. Exposure tests were therefore carried out on specimens

TABLE 8.—*Protective effect of water-gas tar applied to concrete by brush coating. Specimens stored in sulphate solution, major group 3, cement A*[1]

Subgroup No.	Crate No.	W/C ratio	Cured—days		Brush coats of water-gas tar	Water-gas tar absorbed	Percentage of gain in weight after laboratory storage						Number of lake samples attacked and failed after—									
			Moist	Air			¼ month	4 to 6 months	15 to 17 months	23 to 29 months[2]	38 to 40 months	51 months	14 months	26 months	38 months	50 months	62 months	74 months	86 months	98 months	110 months	121 months
17	154	0.8	7	1	4	0.86	0.68	1.31	1.90	F (29)			0	6	6	2F	F					
					6	1.12	.51	1.10	1.80	F (29)			0	6	6	1F	4F	F				
Average						.99	.59	1.20	1.85													
18	155	.8	7	7	4	.99	1.09	1.92	5.64	F (23)			6	6	F							
					6	1.44	.89	1.68	5.45	F (23)			6	6	F							
Average						1.22	.99	1.80	5.54													
19	156	.8	28	1	4	.73	.67	1.23	2.11	F (28)			0	6	F							
					6	.97	.55	1.09	1.74	F (28)			0	6	6	6	F					
Average						.85	.61	1.16	1.92													
20	157	.8	28	7	4	.59	1.19	1.96	2.40	F (27)			0	6	6	3F	F					
					6	.80	.84	1.72	2.16	F (27)			0	6	6	2F	F					
Average						.70	1.01	1.84	2.28													
21	158	.7	7	1	4	.85	.82	1.72	2.37	2.51 (28)	F		0	6	6	6	F					
					6	1.20	.53	1.24	2.26	2.58 (28)	F		0	6	6	6	6	6	F			
Average						1.03	.67	1.48	2.32	2.54												
22	159	.7	7	7	4	.95	.93	1.65	2.56	2.67 (28)	F		0	6	6	1F	F					
					6	1.33	.82	1.51	2.33	2.44 (28)	F		0	6	6	6	3F	F				
Average						1.14	.87	1.58	2.44	2.56												
23	160	.7	28	1	4	.40	.79	1.41	1.83	2.20 (29)	2.62	F	0	6	6	6	4F	F				
					6	.52	.56	1.11	1.24	1.56 (29)	1.79	F	0	6	6	6	2F	F				
Average						.46	.67	1.26	1.53	1.88	2.20											
24	161	.7	28	7	4	.46	.75	1.15	1.55	1.90 (27)	2.41	F	0	0	0	0	2	F				
					6	.57	.73	.99	1.56	1.98 (27)	2.11	F	0	0	0	0	2	2	4	1F	5F	F
Average						.52	.74	1.07	1.55	1.94	2.26											

[1] Values based on 3 and 6 specimens for laboratory and lake storage respectively. F indicates failure of specimens.

[2] Figures in parentheses indicate number of months specimens had been in solution at time of weighing, or number of months in solution until failure.

treated with this material alone and in combination with water-gas tar and seal coats of coal tar in the manner indicated in table 4. The concrete test specimens were fabricated both with low alkali-resisting cement A and with high alkali-resisting cement B.

As the exposure tests progressed, it became evident that the alkali resistance did not materially increase with the quantity of fluosilicate applied, but was affected more by the curing of the test samples. For this reason the results of individual treatments have been combined and the values recorded in tables 10 and 11 are the averages of all treatments applied to each subgroup of specimens.

The laboratory test results indicate that none of the treatments involving the application of ammonium fluosilicate to concrete made with cement A (table 10) was particularly effective.

The specimens of major group 5, treated with fluosilicate by immersion, showed no better resistance than similar untreated specimens (table 5) while, with the exception of major group 7, few of the laboratory specimens attained ages greater than 20 months. The lake specimens showed about the same general trend of performance as that for the laboratory specimens. Major group 7, treated by immersion with fluosilicate and water-gas tar, has shown the best resistance to sulphate action, both in the laboratory and in the lake, but even in this group the performance is not impressive as compared with that of the specimens treated with water-gas tar and given a seal coat of coal tar. (See tables 7 and 9.)

Table 11 shows the results obtained with the various treatments with ammonium fluosilicate and tar applied to concrete made with cement B. In view of the results shown in table 10 for concrete made with cement A, it must be concluded that the greatly superior resistance recorded in table 11 is due primarily to the character of the cement rather than to the protective treatments. It is also concluded that ammonium fluosilicate, either alone or in the various combinations with water-gas tar and coal tar, does not afford effective protection to concrete exposed to the action of sulphate waters.

Photographs of representative crates of specimens from major group 7, made with cements A and B, are shown in figure 4. Two crates from major group 5 made with cement B are shown in figure 5.

PROTECTIVE VALUE OF SODIUM HYDROXIDE

The specimens in major groups 5–S to 10–S, inclusive, were made with cement A and the treatments were identical with those given the specimens in the corresponding major groups 5 to 10, inclusive, except that a 10-percent solution of sodium hydroxide was substituted for the 20-percent solution of ammonium fluosilicate. The results of the exposure tests, which were made only in the laboratory, are shown in table 12.

TABLE 9.—*Protective effect of water-gas tar applied to concrete by brush coating, followed by a seal coat of coal tar. Specimens stored in sulphate solution, major group 4, cement A*[1]

Sub-group No.	Crate No.	W/C ratio	Cured—days		Brush coats of water-gas tar	Percentage of tar absorbed		Percentage of gain in weight after laboratory storage								Number of lake samples attacked and failed after									
			Moist	Air		Water-gas tar	Coal tar	¼ month	4 to 6 months	51 or 52 months	62 to 64 months	74 to 76 months	99 or 100 months	110 to 112 months	129 months	14 months	26 months	38 months	50 months	62 months	74 months	86 months	98 months	110 months	121 months
25	154	0.8	7	1	4	0.86	0.30	0.32	0.87	1.31	1.71	1.82	2.85	F		0	0	0	0	2	6	F			
					6	1.11	.31	.19	.61	1.29	1.75	3.06	F			0	0	0	0	1	6	F			
Average						.99	.30	.26	.74	1.30	1.73	2.44													
26	155	.8	7	7	4	.99	.27	.46	1.10	3.82	F					0	0	3	3	3	2F	F			
					6	1.44	.27	.38	.98	2.12	2.82	4.20	F			0	0	0	1	1	6	F			
Average						1.22	.27	.42	1.04	2.97	4.07														
27	156	.8	28	1	4	.74	.30	.42	1.08	F						0	0	0	0	3	6	6	F		
					6	.97	.30	.25	.90	F						0	0	0	0	2	6	6	F		
Average						.85	.30	.33	.99	...															
28	157	.8	28	7	4	.59	.31	.67	1.56	2.54	3.00	3.79	4.69	F		0	0	0	0	1	6	1F	F		
					6	.80	.31	.55	1.32	2.00	2.18	2.29	4.51	F		0	0	0	0	1	6	6	F		
Average						.70	.31	.61	1.44	2.27	2.59	3.04	4.60												
29	158	.7	7	1	4	.86	.28	.23	.80	1.28	1.32	1.55	F			0	0	0	0	1	6	6	6	1F	[illegible]
					6	1.20	.28	.13	.84	1.32	1.32	1.51	F			0	0	0	0	1	6	6	6	[illegible]	2F
Average						1.03	.28	.18	.82	1.30	1.32	1.53													
30	159	.7	7	7	4	.94	.34	.35	.88	1.42	1.44	1.56	1.93	2.01	2.01	0	0	0	0	2	6	6	6	3F	F
					6	1.33	.34	.25	.90	1.44	1.50	1.55	1.88	2.17	2.17	0	0	0	0	2	6	6	1F	1F	F
Average						1.14	.34	.30	.89	1.43	1.47	1.55	1.90	2.10	2.10										
31	160	.7	28	1	4	.40	.37	.31	.90	1.19	1.33	1.44	2.10	F		0	0	0	0	1	6	6	1F	F	
					6	.53	.37	.23	.80	1.08	1.27	1.35	1.89	2.37	3.28	0	0	0	0	1	6	6	2F	F	
Average						.46	.37	.27	.85	1.14	1.30	1.39	1.99												
32	161	.7	28	7	4	.46	.39	.59	.98	1.77	1.96	2.27	5.26	F		0	0	0	0	0	0	0	[illegible]	[illegible]	1F
					6	.57	.39	.33	.80	1.60	1.70	2.14	F			0	0	0	0	0	0	0	0	2	2F
Average						.52	.39	.46	.89	1.68	1.83	2.20													

[1] Values are based on 3 and 6 specimens for laboratory and lake storage respectively. F indicates failure of specimens.

The protection afforded by the treatments involving the use of sodium hydroxide was, with some exceptions, generally superior to that afforded by the similar treatments with ammonium fluosilicate. It is perhaps noteworthy that the greatest resistance to attack was shown by the specimens of major groups 6–S and 9–S which were the only ones with a seal coat of coal tar. However, the protection afforded by the sodium hydroxide treatments was distinctly inferior to the protective effect of the treatment with water-gas tar followed by a seal coat of coal tar as recorded in tables 7 and 9.

CONCLUSIONS

From the data that have been presented, the following conclusions are drawn:

1. The trends of performance of the specimens exposed in Medicine Lake and those exposed to the action of sulphate solutions in the laboratory were in sufficiently close agreement to justify the conclusion that the results of laboratory tests of this character may be considered indicative of the performance to be expected under natural conditions of exposure.

2. Concrete fabricated with the cement having a low ratio of alumina to iron was more resistant to sulphate action than was concrete containing the cement having a high ratio.

3. The application of water-gas tar to concrete either by immersion or by brush coating afforded about the same degree of protection. The resistance to sulphate action was greatly increased by the addition of a seal coat of coal tar. The data indicated no appreciable added benefit from immersion in water-gas tar for more than 1 hour or from the application of more than 4 brush coats.

4. Ammonium fluosilicate, alone or in combination with water-gas tar or coal tar, did not afford adequate protection from the action of sulphate waters.

5. Sodium hydroxide, alone or in combination with water-gas tar or coal tar, afforded better protection than similar treatments with ammonium fluosilicate. However, all the treatments with sodium hydroxide were distinctly inferior to the treatment with water-gas tar followed by a seal coat of coal tar.

6. The resistance of the treated concrete was influenced by the character of the concrete and by its curing prior to treatment. Concrete of the richer mix having the lower water-cement ratio was more resistant than concrete of the leaner mix with the higher water-cement ratio. Also concrete cured for a short period (1 day) in air prior to treatment was more resistant than concrete cured for a longer period (7 days).

TABLE 10.—*Protective effect of ammonium fluosilicate and tar on*

Major group No.	Sub-group No.	Crate No.	W/C ratio	Cured		Treatment	Average absorbed		Gain in weight after laboratory storage		
				Moist	Air		Salt	Tar	¼ month	3 to 10 months	11 to 20 months
				Days	*Days*		*Percent*	*Percent*	*Percent*	*Percent*[1]	*Percent*[1]
5	1	202	0.8	7	7	Immersion in ammonium fluosilicate for periods of ½, 1, 4, and 24 hours.	1.32	......	2.78	F (10)	......
	2	203	.8	7	28		2.00	......	3.36	F (9)	......
	3	204	.8	28	7		1.04	......	1.90	F (8)	......
	4	205	.8	28	28		1.17	......	2.91	F (8)	......
	5	206	.7	7	7		1.14	......	2.28	F (9)	......
	6	207	.7	7	28		1.47	......	3.37	F (8)	......
	7	208	.7	28	7		.75	......	1.84	F (8)	......
	8	209	.7	28	28		1.05	......	2.45	F (7)	......
						Group average	1.24	......	2.61	......	......
6	9	210	.8	7	7	Immersion in ammonium fluosilicate for periods of ½, 1, 4, and 24 hours plus 1 brush coat of coal tar.	1.76	0.45	1.95	2.52 (6)	F (19)
	10	211	.8	7	28		2.22	.51	1.68	3.02 (5)	F (20)
	11	212	.8	28	7		.98	.50	1.35	1.88 (5)	F (18)
	12	213	.8	28	28		1.33	.42	1.62	2.98 (4)	F (19)
	13	214	.7	7	7		1.38	.49	1.69	2.47 (5)	F (18)
	14	215	.7	7	28		1.49	.37	1.69	3.05 (5)	F (17)
	15	216	.7	28	7		.70	.35	.82	2.09 (4)	F (17)
	16	217	.7	28	28		1.03	.42	1.37	2.36 (4)	F (16)
						Group average	1.36	.44	1.52	2.55	......
7	17	219	.8	7	7	Immersion in ammonium fluosilicate for periods of ½, 1, 4, and 24 hours followed by equal periods of immersion in water-gas tar.	1.80	.53	1.58	1.66 (5)	1.70 (15)
	18	220	.8	7	28		2.49	.61	2.52	2.45 (5)	2.50 (15)
	19	221	.8	28	7		.84	.34	1.30	1.62 (4)	2.28 (14)
	20	222	.8	28	28		1.26	.41	1.52	1.86 (3)	2.53 (13)
	21	223	.7	7	7		1.38	.42	1.29	1.40 (5)	1.65 (15)
	22	224	.7	7	28		1.85	.57	1.44	1.83 (4)	1.90 (14)
	23	225	.7	28	7		.81	.37	.90	1.17 (3)	1.70 (11)
	24	226	.7	28	28		.83	.45	1.56	1.98 (3)	2.55 (11)
						Group average	1.41	.46	1.50	1.75	2.10
8	25	227	.8	7	7	4 and 6 brush coats of ammonium fluosilicate.	2.00	......	2.58	3.19 (7)	5.08 (19)
	26	228	.8	7	28		1.84	......	3.49	4.09 (7)	F (18)
	27	229	.8	28	7		1.43	......	1.78	2.08 (6)	F (18)
	28	230	.8	28	28		1.52	......	2.73	3.08 (5)	F (17)
	29	231	.7	7	7		1.67	......	1.96	2.33 (7)	2.81 (18)
	30	232	.7	7	28		1.55	......	2.78	3.26 (7)	3.85 (18)
	31	233	.7	28	7		1.33	......	1.97	2.26 (6)	F (17)
	32	234	.7	28	28		1.31	......	2.29	2.84 (6)	F (17)
						Group average	1.58	......	2.45	2.89	......
9	33	227	.8	7	7	4 and 6 brush coats of ammonium fluosilicate plus 1 brush coat of coal tar.	2.06	.43	2.47	3.01 (7)	F (19)
	34	228	.8	7	28		1.78	.39	1.90	3.51 (7)	F (18)
	35	229	.8	28	7		1.40	.42	1.26	1.97 (6)	4.59 (18)
	36	230	.8	28	28		1.42	.39	1.17	2.78 (5)	F (17)
	37	231	.7	7	7		1.68	.39	1.63	2.12 (7)	2.34 (18)
	38	232	.7	7	28		1.60	.44	1.43	2.63 (7)	3.31 (18)
	39	233	.7	28	7		1.28	.39	1.22	2.06 (6)	F (17)
	40	234	.7	28	28		1.28	.36	1.21	2.27 (6)	F (17)
						Group average	1.56	.40	1.54	2.54	......
10	41	235	.8	7	7	4 and 6 brush coats of ammonium fluosilicate plus an average of 3 brush coats of water-gas tar.	1.41	.67	1.48	3.71 (10)	F (20)
	42	236	.8	7	28		1.86	.85	1.65	2.56 (9)	F (20)
	43	237	.8	28	7		1.56	.48	1.11	1.75 (9)	F (20)
	44	238	.8	28	28		1.52	.40	1.43	2.64 (8)	F (20)
	45	239	.7	7	7		1.40	.50	1.21	1.95 (10)	F (20)
	46	240	.7	7	28		1.72	.79	1.10	1.97 (9)	F (20)
	47	241	.7	28	7		1.27	.32	.71	1.16 (8)	1.54 (19)
	48	242	.7	28	28		1.45	.38	.93	1.92 (7)	......
						Group average	1.52	.55	1.20	2.21	......

[1] Values for major groups 5, 6, and 7 are based on 12 specimens for laboratory and 24 specimens for lake storage. Values in major groups 8, 9, and 10 are for 6 specimens for laboratory storage and 12 specimens for lake storage for groups 8 and 9, and 24 specimens for lake storage for group 10. F indicates failure of specimens. Figures in parentheses indicate number of months specimens were in solution at time of weighing, or number of months in solution at time of failure.

concrete made with cement A, stored in sulphate solution [1]

Gain in weight after laboratory storage				Number of lake samples attacked and failed after—									
24 to 27 months	30 or 31 months	38 or 39 months	61 months	3 to 8 months	11 to 13 months	15 to 21 months	23 to 25 months	27 to 33 months	35 to 45 months	47 to 53 months	59 to 65 months	72 months	84 months
Percent [1]	*Percent*	*Percent*	*Percent*										
				0 (8)		F (20)							
				0 (7)		0 (19)		F (32)					
				0 (7)		24 (19)		F (31)					
				0 (6)		F (18)							
				0 (8)		18 (20)		F (32)					
				0 (7)		0 (19)		F (32)					
				0 (7)		0 (19)		F (30)					
				0 (6)		0 (18)		F (31)					
				0 (5)		12 (17)		F (30)					
				0 (4)		F (16)							
				0 (4)		4 (16)		12 (28)	F (40)				
				0 (3)		0 (15)		3 (27)	9F (39)	F (52)			
				0 (4)		2 (16)		24 (28)	2F (40)	F (53)			
				0 (4)		0 (16)		6 (28)	18 (40)	18F (53)	F (65)		
				0 (4)		0 (16)		5 (28)	1F (40)	12F (53)	F (65)		
				0 (3)		0 (15)		4 (27)	1F (39)	F (52)			
2.80 (26)		F			0 (13)		4 (25)		18 (37)	13F (52)	F (64)		
2.93 (26)		F			0 (13)		2 (25)		24 (37)	24 (52)	F (64)		
F (25)					0 (13)		10 (25)		17F (37)	F (49)			
F (24)					0 (12)		0 (24)		14 (36)	6F (48)	F (60)		
F (27)					0 (13)		0 (25)		29 (38)	18F (50)	F (62)		
2.11 (25)		2.59	F		0 (12)		0 (24)		4 (36)	4 (48)	4 (60)	3F (72)	12F
F (24)					0 (12)		0 (24)		18 (36)	18F (48)	F (60)		
F (25)					0 (11)		0 (23)		13 (35)	6F (47)	F (59)		
	F			0 (6)		1 (18)		6F (30)	F (42)				
				0 (6)		3 (18)		F (30)					
				0 (5)		6 (17)		F (29)					
				0 (5)		3 (17)		F (29)					
	F			0 (5)		0 (17)		3F (29)	F (41)				
	F			0 (5)		0 (17)		2F (29)	F (41)				
				0 (4)		0 (16)		F (28)					
				0 (4)		0 (16)		F (28)					
				0 (6)		1 (18)		F (30)					
				0 (6)		3 (18)		F (30)					
	F			0 (5)		4 (17)		4F (29)	F (41)				
				0 (5)		2 (17)		6F (29)	F (41)				
	F			0 (5)		0 (17)		1 (29)	F (41)				
	F			0 (5)		0 (17)		6 (29)	6F (41)	F (53)			
				0 (4)		0 (16)		12 (28)	12 (40)	F (52)			
				0 (4)		0 (16)		12 (28)	F (40)				
				0 (8)		2 (21)		18F (33)	F (45)				
				0 (8)		12F (20)		F (32)					
				0 (7)		0 (19)		12F (31)	F (43)				
				0 (6)		0 (18)		12F (30)	F (42)				
				0 (8)		0 (20)		12F (32)	F (44)				
				0 (7)		2 (19)		F (31)					
	F			0 (6)		0 (18)		0 (30)	21 (42)	F (54)			
F				0 (5)		0 (17)		6 (29)	F (41)				

TABLE 11.—*Protective effect of ammonium fluosilicate and tar on*

Major group No.	Sub-group No.	Crate No.	W/C ratio	Cured		Treatment	Average absorbed		Gain in weight		
				Moist	Air		Salt	Tar	¼ month	37 to 46 months	48 to 58 months
				Days	*Days*		*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
5	1	162	0.8	7	7	Immersion in ammonium fluosilicate for periods of ½, 1, 4, and 24 hours.	1.52	....	2.80	3.64 (45)	4.22 (55)
	2	163	.8	7	28		2.33	....	3.37	4.49 (44)	6F (55)
	3	164	.8	28	7		.97	....	2.29	2.99 (44)	3.14 (56)
	4	165	.8	28	28		1.13	....	3.17	4.05 (44)	4.46 (55)
	5	166	.7	7	7		1.09	....	2.30	2.97 (45)	3.11 (56)
	6	167	.7	7	28		1.66	....	3.16	3.74 (44)	4.24 (55)
	7	168	.7	28	7		.66	....	1.68	2.46 (43)	2.73 (54)
	8	169	.7	28	28		.76	....	2.29	3.12 (43)	3.30 (54)
						Group average	1.26	....	2.63	3.43	3.79
6	9	170	.8	7	7	Immersion in ammonium fluosilicate for periods of ½, 1, 4, and 24 hours plus 1 brush coat of coal tar.	2.08	0.37	2.44	3.20 (44)	3.20 (55)
	10	171	.8	7	28		3.07	.27	1.37	3.84 (42)	3.99 (52)
	11	172	.8	28	7		1.13	.28	.68	2.17 (42)	2.17 (53)
	12	173	.8	28	28		1.65	.34	1.17	2.95 (41)	2.95 (52)
	13	174	.7	7	7		1.25	.36	1.15	2.31 (42)	2.31 (54)
	14	175	.7	7	28		1.80	.31	1.41	2.80 (40)	2.80 (52)
	15	176	.7	28	7		.58	.30	.65	1.88 (41)	1.88 (53)
	16	177	.7	28	28		.87	.30	.46	2.29 (41)	2.35 (52)
						Group average	1.55	.32	1.17	2.68	2.71
7	17	178	.8	7	7	Immersion in ammonium fluosilicate for periods of ½, 1, 4 and 24 hours followed by equal periods of immersion in water-gas tar.	2.72	1.50	1.50	2.24 (40)	2.24 (52)
	18	179	.8	7	28		3.11	.97	2.10	2.71 (38)	2.71 (50)
	19	181	.8	28	7		.83	.72	1.57	2.14 (44)	2.26 (52)
	20	180	.8	28	28		1.38	.72	2.08	2.78 (37)	2.81 (48)
	21	182	.7	7	7		1.69	.95	1.24	1.85 (38)	1.96 (50)
	22	183	.7	7	28		2.27	.62	2.35	2.91 (37)	2.91 (49)
	23	184	.7	28	7		.77	.60	1.39	2.02 (45)	2.06 (56)
	24	185	.7	28	28		.89	.55	2.00	2.29 (43)	2.29 (55)
						Group average	1.71	.83	1.78	2.36	2.40
8	25	186	.8	7	7	4 and 6 brush coats of ammonium fluosilicate.	3.52	....	3.43	4.30 (46)	4.51 (58)
	26	187	.8	7	28		3.28	....	4.70	F (43)	....
	27	188	.8	28	7		1.40	....	2.28	2.50 (43)	2.80 (55)
	28	189	.8	28	28		1.62	....	3.12	3.39 (43)	3.95 (55)
	29	190	.7	7	7		3.18	....	3.20	3.53 (41)	3.77 (53)
	30	191	.7	7	28		2.67	....	3.51	4.10 (41)	4.36 (53)
	31	192	.7	28	7		1.70	....	2.33	2.64 (42)	2.75 (54)
	32	193	.7	28	28		1.92	....	2.68	3.09 (42)	3.24 (54)
						Group average	2.41	....	3.15	....	....
9	33	186	.8	7	7	4 and 6 brush coats of ammonium fluosilicate plus 1 brush coat of coal tar.	3.37	.44	3.03	4.26 (45)	4.36 (57)
	34	187	.8	7	28		3.43	.51	4.15	F (44)	....
	35	188	.8	28	7		1.25	.48	.95	1.91 (43)	1.91 (55)
	36	189	.8	28	28		1.62	.50	1.32	2.90 (42)	2.96 (55)
	37	190	.7	7	7		2.91	.44	3.13	3.47 (41)	3.46 (53)
	38	191	.7	7	28		2.71	.41	3.22	3.86 (41)	3.86 (53)
	39	192	.7	28	7		1.71	.41	1.21	1.99 (42)	2.09 (54)
	40	193	.7	28	28		1.90	.55	.97	2.68 (42)	2.77 (54)
						Group average	2.36	.47	2.25	....	....
10	41	194	.8	7	7	4 and 6 brush coats of ammonium fluosilicate plus an average of 2 brush coats of water-gas tar.	3.59	1.02	2.28	2.69 (41)	2.76 (53)
	42	195	.8	7	28		3.19	.53	3.45	3.73 (39)	3.75 (52)
	43	196	.8	28	7		2.13	.20	1.65	2.19 (38)	2.19 (51)
	44	197	.8	28	28		2.05	.25	3.07	3.40 (37)	3.40 (50)
	45	198	.7	7	7		3.19	.34	2.13	2.64 (39)	2.72 (52)
	46	199	.7	7	28		2.26	.40	2.58	3.01 (38)	3.06 (51)
	47	200	.7	28	7		1.67	.25	1.03	1.56 (37)	1.63 (50)
	48	201	.7	28	28		2.57	.28	1.93	2.61 (37)	2.61 (49)
						Group average	2.58	.41	2.51	2.73	2.76

[1] Values for major groups 5, 6, and 7 are based on 12 specimens for laboratory and 24 specimens for lake storage. Values in major groups 8, 9, and 10 are for 6 specimens for laboratory storage and 12 specimens for lake storage in the case of major groups 8 and 9, and 24 specimens for lake storage for major group 10. F indicates failure of specimens. Figures in parentheses indicate number of months specimens were in solution at time of weighing, or number or months in solution at time of failure.

concrete made with cement B and stored in sulphate solution [1]

after laboratory storage				Number of lake specimens attacked and failed after—									
60 to 69 months	73 to 93 months	85 to 105 months	102 to 106 months	1 to 11 months	12 to 23 months	24 to 35 months	36 to 47 months	48 to 59 months	61 to 71 months	72 to 83 months	84 to 95 months	96 to 101 months	102 to 113 months
Percent	*Percent*	*Percent*	*Percent*										
4.77 (69)	5.63 (93)	F (105)	-----	0 (7)	0 (19)	24 (31)	24 (43)	24 (55)	24 (67)	18F (79	F (91)	-----	-----
7F (67)	F (92)	-----	-----	0 (6)	0 (18)	24 (30)	F (42)	-----	-----	-----	-----	-----	-----
3.45 (69)	3F (93)	6F (105)	6F (122)	0 (7)	0 (19)	12 (31)	18 (43)	24 (55)	24 (67)	12F (79)	F (91)	-----	-----
5.47 (68)	F (92)	-----	-----	0 (6)	0 (18)	0 (30)	9 (42)	24 (54)	6F (66)	6F (78)	F (90)	-----	-----
3.50 (68)	F (92)	-----	-----	0 (8)	0 (20)	12 (32)	24 (44)	24 (56)	24 (68)	12F (80)	F (92)	-----	-----
5.02 (68)	F (92)	-----	-----	0 (6)	0 (18)	12 (30)	18 (42)	24 (54)	24 (66)	24 (78)	F (90)	-----	-----
2.89 (67)	6F (91)	6F (103)	6F (120)	0 (6)	0 (18)	6 (30)	6 (42)	24 (54)	24 (66)	24 (78)	F (90)	-----	-----
3.40 (67)	F (91)	-----	-----	0 (6)	0 (18)	0 (30)	0 (42)	24 (54)	24 (66)	24 (78)	F (90)	-----	-----
4.21	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
3.20 (67)	3.68 (91)	3.81 (103)	3.81 (121)	0 (5)	0 (17)	0 (29)	5 (41)	6 (53)	24 (65)	24 (77)	1F (89)	1F (101)	1F (113)
4.96 (65)	6F (89)	6F (101)	7F (118)	0 (3)	0 (15)	0 (27)	7 (39)	24 (51)	6F (63)	F (75)	-----	-----	-----
2.17 (66)	2.43 (90)	2.55 (101)	2.55 (119)	0 (4)	0 (16)	0 (28)	3 (40)	3 (52)	4 (64)	4 (76)	4 (88)	4 (100)	4 (112)
3.37 (65)	3.58 (89)	3.81 (101)	3.81 (119)	0 (3)	0 (15)	0 (27)	0 (39)	5 (51)	24 (63)	24 (75)	18F (87)	5F (99)	-----
2.39 (66)	2.61 (90)	2.73 (102)	2.73 (119)	0 (4)	0 (16)	0 (28)	2 (40)	2 (52)	5 (64)	5 (76)	5 (88)	5 (100)	24 (112)
2.84 (64)	3.19 (89)	3.34 (101)	3.34 (116)	0 (3)	0 (15)	0 (27)	3 (39)	6 (51)	6 (63)	6 (75)	21 (87)	3F (99)	4F (111)
1.94 (65)	2.16 (89)	2.26 (101)	2.26 (119)	0 (4)	0 (16)	0 (28)	1 (40)	1 (52)	1 (64)	1 (76)	1 (88)	1 (100)	1 (112)
2.48 (65)	2.71 (89)	2.79 (101)	2.79 (119)	0 (3)	0 (15)	0 (27)	0 (39)	1 (51)	1 (63)	1 (75)	1 (87)	2 (99)	11 (110)
2.92	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
2.32 (64)	2.64 (88)	2.73 (100)	2.73 (117)	0 (2)	0 (14)	0 (26)	0 (38)	6 (50)	15 (62)	15 (74)	24 (86)	24 (98)	1F (110)
2.71 (62)	2.92 (87)	2.98 (99)	2.98 (116)	0 (1)	0 (13)	0 (25)	0 (37)	0 (49)	0 (61)	0 (73)	1 (85)	1 (97)	2 (109)
2.26 (62)	2.85 (80)	3.08 (92)	3.08 (110)	0 (7)	0 (19)	0 (31)	0 (43)	12 (55)	15 (67)	11F (79)	18F (91)	-----	18F (103)
2.97 (60)	3.25 (85)	3.41 (97)	3.41 (114)	0 (11)	0 (23)	0 (35)	0 (47)	0 (59)	0 (71)	1 (83)	3 (95)	-----	10 (107)
1.99 (63)	2.25 (87)	2.43 (99)	2.43 (116)	0 (1)	0 (13)	0 (25)	0 (37)	0 (49)	0 (61)	2 (73)	6 (85)	18 (97)	18 (109)
2.91 (61)	3.17 (86)	3.31 (98)	3.31 (115)	0 (1)	0 (12)	0 (24)	0 (36)	1 (48)	1 (60)	1 (72)	1 (84)	1 (96)	1 (108))
-----	2.34 (80)	2.48 (92)	2.48 (110)	0 (7)	0 (18)	0 (30)	0 (42)	9 (54)	12 (66)	18 (78)	24 (90)	-----	24 (103)
2.49 (68)	2.91 (80)	3.06 (92)	3.06 (109)	0 (6)	0 (18)	0 (30)	1 (42)	7 (54)	9 (66)	18 (78)	24 (90)	-----	24 (102)
-----	2.79	2.93	2.93	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
-----	3F (81)	3F (93)	F (111)	0 (8)	0 (20)	0 (32)	12 (44)	F (56)	-----	-----	-----	-----	-----
-----	-----	-----	-----	0 (6)	0 (18)	0 (30)	12 (42)	F (54)	-----	-----	-----	-----	-----
-----	3.55 (79)	3.74 (91)	F (108)	0 (5)	0 (17)	0 (29)	0 (41)	6 (53)	6 (65)	F (77)	-----	-----	-----
-----	F (79)	-----	-----	0 (5)	0 (17)	0 (29)	12 (41)	6F (53)	F (65)	-----	-----	-----	-----
-----	4 24 (77)	4.34 (89)	4.34 (106)	0 (3)	0 (15)	0 (27)	0 (39)	0 (51)	6 (63)	6 (75)	6F (87)	6F (99)	-----
-----	3F (76)	3F (88)	F (106)	0. (4)	0 (16)	0 (28)	12 (40)	12 (52)	F (64)	-----	-----	-----	-----
-----	3.18 (78)	3.31 (90)	3.31 (107)	.0 (4)	0 (16)	0 (28)	0 (40)	0 (52)	12 (64)	12 (76)	12 (88)	5F (100)	-----
-----	3F (78)	3F (90)	F (107)	0 (5)	0 (17)	0 (29)	12 (41)	12 (53)	12 (65)	F (77)	-----	-----	-----
-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
-----	4.65 (81)	4.79 (93)	4.81 (111)	0 (8)	0 (20)	0 (32)	0 (44)	0 (56)	0 (68)	6 (80)	12 (92)	-----	12 (104)
-----	-----	-----	-----	0 (6)	0 (18)	0 (30)	6 (42)	12 (54)	12 (66)	F (78)	-----	-----	-----
-----	1.99 (79)	2.11 (91)	2.11 (108)	0 (5)	0 (17)	0 (29)	0 (41)	0 (53)	0 (65)	0 (77)	0 (89)	0 (101)	-----
-----	3.37 (79)	3.49 (91)	3.49 (108)	0 (5)	0 (17)	0 (29)	0 (41)	4 (53)	12 (65)	F (77)	-----	-----	-----
-----	3.67 (77)	3.78 (89)	3.78 (106)	0 (3)	0 (15)	0 (27)	0 (39)	0 (51)	0 (63)	0 (75)	0 (87)	6 (99)	-----
-----	4.10 (76)	4.17 (88)	4.17 (106)	0 (4)	0 (16)	0 (28)	0 (40)	0 (52)	0 (64)	0 (76)	2 (88)	11 (100)	-----
-----	2.27 (78)	2.40 (90)	2.40 (107)	0 (4)	0 (16)	0 (28)	0 (40)	0 (52)	0 (64)	0 (76)	0 (88)	0 (100)	-----
-----	3.01 (78)	3.08 (90)	3.08 (107)	0 (5)	0 (17)	0 (29)	0 (41)	0 (53)	0 (65)	0 (77)	0 (89)	7 (101)	-----
-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----
-----	3.09 (77)	3.23 (89)	3.23 (106)	0 (3)	0 (15)	0 (27)	0 (39)	2 (51)	18 (63)	12F (75)	12F (87)	F (99)	-----
-----	4.15 (76)	4.34 (88)	4.34 (105)	0 (2)	0 (14)	0 (26)	0 (38)	2 (50)	6F (62)	18F (74)	18F (86)	F (98)	-----
-----	2.27 (75)	2.40 (87)	2.47 (104)	0 (1)	0 (13)	0 (25)	0 (37)	0 (49)	0 (61)	0 (72)	0 (84)	8 (97)	-----
-----	3.67 (74)	3.80 (86)	3.80 (103)	0 (1)	0 (12)	0 (24)	0 (36)	0 (48)	0 (60)	0 (72)	0 (84)	0 (96)	-----
-----	3.02 (76)	3.15 (88)	3.15 (105)	0 (2)	0 (14)	0 (26)	0 (38)	0 (50)	0 (62)	3 (74)	11 (86)	24 (98)	-----
-----	3.30 (75)	3.42 (87)	3.42 (104)	0 (1)	0 (13)	0 (25)	0 (37)	0 (49)	6 (61)	12 (72)	6F (84)	F (97)	-----
-----	1.90 (74)	2.04 (86)	2.04 (103)	0 (1)	0 (12)	0 (24)	0 (36)	0 (48)	0 (60)	0 (72)	0 (84)	12 (96)	-----
-----	2.76 (73)	2.94 (85)	2.94 (102)	0 (11)	0 (23)	0 (35)	0 (47)	0 (59)	0 (71)	0 (83)	0 (95)	-----	-----
-----	3.02	3.16	3.17	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----

TABLE 12.—*Protective effect of sodium hydroxide on concrete made with cement A and stored in sulphate solution* [1]

Major group No.	Sub-group No.	W/C Ratio	Cured		Treatment	Average absorbed		Gain in weight after laboratory storage							
			Moist	Air		Sodium hydroxide	Tar	¼ month	2 to 9 months	15 to 21 months	27 to 29 months	40 or 41 months	51 to 64 months	64 to 76 months	93 months
			Days	*Days*		*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
5-S	1	0.8	7	7	Immersion in sodium hydroxide for periods of ½, 1, 4, and 24 hours.	1.95	------	2.03	2.99 (6)	F (19)	------	------	------	------	------
	2	.8	7	28		2.13	------	2.72	4.22 (6)	F (19)	------	------	------	------	------
	3	.8	28	7		1.00	------	1.66	2.57 (6)	F (19)	------	------	------	------	------
	4	.8	28	28		1.42	------	2.02	5.22 (6)	F (19)	------	------	------	------	------
	5	.7	7	7		1.50	------	1.44	2.58 (6)	F (19)	------	------	------	------	------
	6	.7	7	28		1.73	------	2.16	3.19 (6)	F (19)	------	------	------	------	------
	7	.7	28	7		.85	------	1.35	1.90 (6)	F (19)	------	------	------	------	------
	8	.7	28	28		1.19	------	1.55	2.69 (6)	F (18)	------	------	------	------	------
					Group average	1.47	------	1.87	3.17	------	------	------	------	------	------
6-S	9	.8	7	7	Immersion in sodium hydroxide for periods of ½, 1, 4, and 24 hours plus 1 brush coat of coal tar.	1.66	0.47	.69	.92 (3)	1.07 (16)	1.37 (27)	2.06	F (64)	------	------
	10	.8	7	28		2.37	.40	.69	1.15 (3)	1.24 (16)	1.90 (27)	3.63	F (64)	------	------
	11	.8	28	7		1.24	.46	.84	1.21 (3)	1.67 (16)	2.50 (27)	3.26	F (63)	------	------
	12	.8	28	28		1.36	.49	.79	1.33 (3)	1.59 (16)	2.80 (27)	F	------	------	------
	13	.7	7	7		1.30	.43	.66	.88 (3)	1.04 (16)	1.15 (27)	1.40	1.80 (64)	1.96 (76)	3F (93)
	14	.7	7	28		1.82	.40	.81	1.19 (3)	1.28 (16)	1.70 (27)	2.14	2.90 (64)	F (76)	------
	15	.7	28	7		1.03	.49	.73	.97 3)	1.02 (16)	1.25 (27)	2.03	F (63)	------	------
	16	.7	28	28		1.25	.46	.62	.97 (3)	1.01 (16)	1.32 (27)	1.63	F (63)	------	------
					Group average	1.50	.45	.73	1.08	1.24	1.74	------	------	------	------
7-S	17	.8	7	7	Immersion in sodium hydroxide for periods of ½, 1, 4, and 24 hours followed by equal periods of immersion in water-gas tar.	1.55	.58	.69	2.70 (9)	F (20)	------	------	------	------	------
	18	.8	7	28		2.32	.67	.89	2.23 (8)	F (19)	------	------	------	------	------
	19	.8	28	7		1.06	.50	.64	2.37 (8)	F (21)	------	------	------	------	------
	20	.8	28	28		1.30	.33	.91	3.15 (8)	F (20)	------	------	------	------	------
	21	.7	7	7		1.17	.49	.48	1.63 (9)	F (20)	------	------	------	------	------
	22	.7	7	28		2.02	.62	.72	1.64 (8)	F (19)	------	------	------	------	------
	23	.7	28	7		.81	.44	.43	1.23 (8)	F (19)	------	------	------	------	------
	24	.7	28	28		1.16	.31	.71	2.00 (8)	F (20)	------	------	------	------	------
					Group average	1.42	.49	.68	2.12	------	------	------	------	------	------
8-S	25	.8	7	7	4 and 6 brush coats of sodium hydroxide.	1.94	------	2.87	3.35 (4)	F (16)	------	------	------	------	------
	26	.8	7	28		1.77	------	3.24	3.76 (4)	F (16)	------	------	------	------	------
	27	.8	28	7		1.26	------	2.10	2.36 (4)	F (16)	------	------	------	------	------
	28	.8	28	28		1.47	------	2.39	3.01 (4)	F (16)	------	------	------	------	------
	29	.7	7	7		1.66	------	2.40	2.80 (4)	F (16)	------	------	------	------	------
	30	.7	7	28		1.59	------	2.34	3.00 (4)	F (19)	------	------	------	------	------
	31	.7	28	7		1.38	------	1.25	1.66 (4)	3.45 (16)	F (28)	------	------	------	------
	32	.7	28	28		1.29	------	1.52	2.08 (4)	4.76 (16)	F (28)	------	------	------	------
					Group average	1.54	------	2.26	2.75	------	------	------	------	------	------
9-S	33	.8	7	7	4 and 6 brush coats of sodium hydroxide plus 1 brush coat of coal tar.	1.93	.44	.58	1.20 (4)	1.48 (16)	1.82 (27)	------	3.02 (52)	F (65)	------
	34	.8	7	28		1.77	.38	.71	1.42 (4)	1.75 (16)	2.18 (29)	------	3.77 (52)	F (65)	------
	35	.8	28	7		1.23	.38	.58	.93 (4)	1.60 (16)	F (28)	------	------	------	------
	36	.8	28	28		1.43	.36	.51	1.26 (4)	2.61 (16)	F (28)	------	------	------	------
	37	.7	7	7		1.63	.41	.42	.93 (4)	1.25 (16)	1.74 (28)	------	2.79 (52)	F (65)	------
	38	.7	7	28		1.54	.38	.66	1.36 (4)	1.70 (16)	2.09 (28)	------	3.47 (52)	F (64)	------
	39	.7	28	7		1.44	.39	.39	.86 (4)	1.05 (16)	1.41 (28)	------	F (52)	------	------
	40	.7	28	28		1.29	.31	.22	.91 (4)	1.82 (16)	1.88 (28)	------	F (51)	------	------
					Group average	1.53	.38	.51	1.11	1.66	------	------	------	------	------
10-S	41	.8	7	7	4 and 6 brush coats of sodium hydroxide plus an average of 4 brush coats of water-gas tar.	1.38	.81	.84	1.66 (5)	F (18)	------	------	------	------	------
	42	.8	7	28		1.48	.85	.90	1.80 (4)	F (17)	------	------	------	------	------
	43	.8	28	7		1.28	.53	.89	1.04 (3)	F (16)	------	------	------	------	------
	44	.8	28	28		1.41	.39	1.77	2.67 (3)	F (16)	------	------	------	------	------
	45	.7	7	7		1.33	.83	.82	1.18 (5)	2.37 (17)	------	F (41)	------	------	------
	46	.7	7	28		1.50	.78	1.19	1.75 (3)	2.67 (16)	------	F (40)	------	------	------
	47	.7	28	7		1.62	.55	.84	1.16 (3)	F (16)	------	------	------	------	------
	48	.7	28	28		1.33	.43	1.20	1.76 (2)	F (15)	------	------	------	------	------
					Group average	1.42	.65	1.06	1.63	------	------	------	------	------	------

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[1] Values are based on 12 specimens for major groups 5-S, 6-S, and 7-S, and on 6 specimens for groups 8-S, 9-S, and 10-S. F indicates failure of specimens. Figures in parentheses indicate number of months specimens were in solution at time of weighing or number of months in solution at time of failure.

DETERMINATION OF CEMENT CONTENT OF SOIL-CEMENT MIXTURES

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by JACOB MIZROCH, Assistant Chemical Engineer

CHEMICAL ANALYSES made during the construction of soil-cement base courses have been of value in locating and controlling discrepancies between required percentages of cement and the amounts actually present. This information is also useful in studies of the pavement during its service life.

In 1939 and 1940 several State Highway Departments began to make chemical analyses as a control measure on soil-cement work. This led to a preliminary survey by Subcommittee No. 2 of the Project Committee on "Stabilized Roads" of the Highway Research Board to determine the methods of analysis in use and their relative merits.

The survey showed that 2 basic methods were in use. At the request of the subcommittee, 23 laboratories cooperated in testing standard soil-cement samples by the 2 methods. The results of these cooperative tests are reported by Carson and Reid in the Proceedings of the Highway Research Board.[1]

Of the 2 basic methods used, method 1 involves determination of the calcium oxide (CaO) content of raw soil, cement, and soil-cement mixtures, and calculation of the percentage of cement in the soil-cement mixtures using the proportionality law applied to mixtures. Method 2 involves determination of the calcium (Ca) content of a sample of soil-cement mixture from the completed roadway and a similar determination on a control sample prepared to contain approximately the same percentage of cement, using the original raw soil and cement as components. From these 2 values the percentage of cement in the roadway sample may be calculated.

Both methods require the removal of silica, iron, and alumina before the subsequent precipitation and determination of calcium. In method 1, three steps are necessary since silica is first filtered off and then iron and alumina are precipitated, filtered off, redissolved, reprecipitated and filtered off a second time. In method 2, the iron and alumina are precipitated in the presence of silica and filtered off together with it, hence saving two steps in the procedure.

In the control of soil-cement construction it is necessary to have a rapid method of determining chemically the cement content of the soil-cement mixtures. From this viewpoint, method 2 is more desirable than method 1 because of the saving in time. However, the subcommittee has recommended method 1 as the more suitable of the two, because of its inherent accuracy and wider applicability.

A procedure combining the accuracy of method 1 and the shortness of method 2 is desirable.

Such a procedure has been worked out at the laboratory of the Public Roads Administration based on the principles brought out by the work of Chapman,[2] and McComas and Rieman[3] on methods of determining calcium in the presence of iron, alumina, and other interfering ions. These investigators found that calcium can be precipitated as the oxalate in the presence of iron, alumina, etc., if the solution is buffered with sufficient ammonium chloride and various organic acids at pH 4.

The Public Roads method, therefore, embodies the following procedure: The samples of soil, soil-cement, and cement are treated with hydrochloric acid, the insoluble residue is filtered off, and calcium is precipitated and determined in the filtrate after sufficiently buffering with ammonium chloride and acetic acid and adjusting the solution to a pH of 4.

Determinations of percentage of cement by this method are fully as accurate as determinations on the same samples by subcommittee method 1. The time required for the analysis by this method is the same as that required by method 2 of the subcommittee. The saving in time over that necessary for subcommittee method 1 amounts to approximately one-half hour per sample.

PREPARATION OF REAGENT SOLUTIONS

Potassium permanganate, 0.1 N.—3.2 gm. $KMnO_4$ is dissolved in 1 l. of distilled water and the solution is heated in a covered beaker for 3 hours at 90° to 95° C., then set aside at room temperature for several days. The solution is then filtered through asbestos and standardized against pure oven-dried sodium oxalate.

Acetic acid, 10 percent.—100 ml. glacial acetic acid is diluted to 1 l. with distilled water.

Oxalic acid, 10 percent.—To 100 gm. oxalic acid is added 400 ml. distilled water and the whole warmed until the crystals dissolve. The solution is filtered into a clean liter flask and diluted to 1 l. with distilled water.

Ammonium chloride, 30 percent.—300 gm. of NH_4Cl is placed in a liter beaker and enough distilled water added to make beaker about four-fifths full. The mixture is stirred, warmed to room temperature and then filtered on 9 cm. grade A, B and A filter paper into a clean liter flask. The undissolved residue is brought on the filter with a stream of distilled water from a wash bottle and dissolved by repeated additions of a few milliliters water to the filter. When all of the salt is in solution, the filtrate is diluted to a liter with distilled water.

Brom-cresol green indicator, 0.04 percent solution. This indicator solution may be obtained from the LaMotte Chemical Co., Baltimore, Md.

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[1] Chemical Determination of Cement Content of Soil-Cement Mixtures from Cement Hardened Bases. By L. S. Carson and Carl Reid. Proceedings, Twenty-first Annual Meeting of the Highway Research Board, p. 471 (1941).

[2] The Precipitation of Calcium Oxalate In the Presence of Iron, Aluminum, Titanium, Manganese, Magnesium, and Phosphorus. H. D. Chapman. Soil Science, vol. 26, p. 479 (1928).

[3] Accurate Determination of Calcium Without Reprecipitation. W. H. McComas and W. Rieman. Industrial and Engineering Chemistry, Analytical Edition, vol. 14, p. 929 (1942).

METHOD OF ANALYSIS DESCRIBED

Soil-cement mixtures.—The soil-cement mixture is air dried, then pulverized to pass the No. 40 sieve. About 50 gm. of the mixture is oven dried at 110° C. to constant weight, then cooled and 1 gm. weighed into a 100 ml. beaker. Ten ml. 1:1 HCl is added, the beaker covered and the solution boiled on a hot plate for 5 minutes. The insoluble residue is filtered off on 7 or 9 cm. grade A, B and A filter paper (or equivalent) and washed chloride-free with hot distilled water (about five washings, after all the residue has been brought on the filter paper) the filtrate being caught in a 250-ml. beaker. Now 10 ml. 30 percent ammonium chloride solution, 10 ml. 10 percent oxalic acid and 10 ml. 10 percent acetic acid are added and the solution diluted to 150 ml.

Ten ml. 30 percent NH_4Cl insures the presence of at least 6 gm. NH_4Cl in the solution when the pH is adjusted to 4. The quantities of reagents given are for soils or soil-cement mixtures containing less than 10 percent acid-soluble iron and aluminum oxides. For soils containing more than 10 percent acid-soluble iron and aluminum oxides, the amounts of reagents given are insufficient to give a satisfactory precipitate of calcium oxalate. When soils with between 10 and 20 percent soluble iron and aluminum oxides were tested, it was found that the quantities of reagents had to be increased to 20 ml. 30 percent NH_4Cl, 15 ml. 10 percent oxalic acid and 15 ml. 10 percent acetic acid. The volume of the solution also had to be increased to 200 ml.

The diluted solution is heated nearly to boiling and 11 ml. 1:1 ammonia is added with stirring. A drop of brom-cresol green indicator is then added. If the drop turns green, the solution is at pH4. If it turns yellow, not enough ammonia had been added; if the drop turns blue, the solution is too basic and a few drops of 5 NHCl must be added till the desired pH is attained. The cloudy solution, now at pH4, is boiled gently for 5 minutes on the hot plate and then set on a cooler portion of the hot plate (or on a steam bath) for an hour for the calcium oxalate to settle out. The calcium oxalate is then filtered off on 7 cm. 00 Munktell filter paper (or equivalent grade), the beaker is rinsed three times with hot distilled water and the precipitate is washed five times with the hot water.

The filter paper is transferred to the rinsed beaker, then 50 ml. hot distilled water, and 5 ml. 1:1 H_2SO_4 are added. The filter paper is opened with a glass rod and the solution is stirred till all the calcium oxalate is in solution. The paper is now lifted with the glass rod, thoroughly rinsed with hot distilled water into the beaker and finally balled up between thumb and forefinger, squeezed dry, and discarded. The solution is heated to between 80° and 90° C. and immediately titrated with 0.1 N standard $KMnO_4$ solution.

$$1 \text{ ml. } 0.1 \text{ N} KMnO_4 = 0.0028 \text{ gm} CaO$$

Untreated soil.—Same procedure is followed as for the soil-cement mixtures.

Cement.—One gm. oven-dried cement is weighed out into a 100 ml. beaker; 50 ml. 1:4 HCl is added and the beaker covered. The solution is boiled 5 minutes on the hot plate and the insoluble residue filtered off, the filtrate being caught in a 250 ml. volumetric flask. When the residue is washed chloride-free the contents of the flask are cooled and the volume adjusted to 250 ml. A 25-ml. aliquot portion is pipetted into a 250-ml. beaker and 4 ml. concentrated HCl added. From this point on the procedure follows exactly the procedure for precipitation and determination of calcium described in the section on soil-cement mixtures.

Calculations:

Let A = percentage CaO in the soil-cement mixture.
B = percentage CaO in the untreated soil.
C = percentage CaO in the cement.

$$\text{Then } \frac{A-B}{C-B} \times 100 = \text{percentage cement in the soil-cement mixture.}$$

Alternate method of calculation:

$$\frac{\text{ml. } KMnO_4 \text{ required for 1 gm. soil-cement} - \text{ml. } KMnO_4 \text{ required for 1 gm. untreated soil}}{\text{ml. } KMnO_4 \text{ required for 1 gm. cement} - \text{ml. } KMnO_4 \text{ required for 1 gm. untreated soil}} \times 100 = \text{percentage cement in soil-cement}$$

This alternate method of calculation has the advantage of doing away with the necessity of standardizing the potassium permanganate solution.

NEW METHOD GIVES SATISFACTORY RESULTS

To test the accuracy of this method, determinations of percentages of cement in soil-cement mixtures were made by this method as well as by that recommended by the subcommittee. Table 1 gives the results on samples obtained by crushing and pulverizing to pass the No. 40 sieve some soil-cement cylinders which had been subjected to 12 cycles of wetting and drying. Table 2 gives the results on samples obtained by crushing and pulverizing air-dry clay-cement cylinders which had been cured in the moist closet for 7 days.

TABLE 1.—*Percentages of calcium oxide and cement, and grams of cement per 100 gm. of soil found by 2 methods*

[Tests made on soil-cement cylinders subjected to 12 cycles of wetting and drying and pulverized to pass the No. 40 sieve]

Sample	CaO		Cement		Cement per 100 gm. of soil	
	Subcommittee method	Public Roads method	Subcommittee method	Public Roads method	Subcommittee method	Public Roads method
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Gm.*	*Gm.*
Soil	0.28	0.29				
5A, 5 gm. cement per 100 gm. soil	3.33	3.33	4.9	4.9	5.2	5.2
7A, 7 gm. cement per 100 gm. soil	4.62	4.59	7.0	7.0	7.6	7.5
9A, 9 gm. cement per 100 gm. soil	5.35	5.29	8.2	8.1	8.9	8.8
5B, 5 gm. cement per 100 gm. soil	3.27	3.32	4.9	4.9	5.2	5.2
7B. 7 gm cement per 100 gm. soil	4.46	4.44	6.8	6.8	7.3	7.3
9B, 9 gm. cement per 100 gm. soil	5.32	5.40	8.2	8.3	8.9	9.0
Cement	61.95	61.75				

TABLE 2.—*Percentages of calcium oxide and cement and grams of cement per 100 gm. clay found by 2 methods.*

[Tests made on air-dry clay-cement cylinders cured in moist closet 7 days and pulverized to pass the No. 40 sieve]

Sample	CaO		Cement		Cement per 100 gm. of clay	
	Subcommittee method	Public Roads method	Subcommittee method	Public Roads method	Subcommittee method	Public Roads method
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Gm.*	*Gm.*
NEW YORK AVENUE CLAY	0.18	0.16	---	---	---	---
3 gm. cement No. 1 per 100 gm. clay	2.02	2.00	2.9	2.9	3.0	3.0
7 gm. cement No. 1 per 100 gm. clay	4.26	4.24	6.5	6.5	6.9	6.9
11 gm. cement No. 1 per 100 gm. clay	6.26	6.23	9.7	9.6	10.7	10.6
Cement No. 1	62.80	63.20	---	---	---	---
3 gm. cement No. 2 per 100 gm. clay	1.93	1.92	2.9	2.9	3.0	3.0
7 gm. cement No. 2 per 100 gm. clay	4.12	4.04	6.4	6.3	6.8	6.7
11 gm. cement No. 2 per 100 gm. clay	5.94	6.02	9.4	9.5	10.4	10.5
Cement No. 2	61.30	61.78	---	---	---	---
3 gm. cement No. 3 per 100 gm. clay	2.04	2.03	2.9	2.9	3.0	3.0
7 gm. cement No. 3 per 100 gm. clay	4.20	4.28	6.2	6.4	6.7	6.8
11 gm. cement No. 3 per 160 gm. clay	6.33	6.32	9.6	9.5	10.6	10.5
Cement No. 3	64.55	64.78	---	---	---	---
3 gm. cement No. 4 per 100 gm. clay	1.94	1.96	2.8	2.9	2.9	3.0
7 gm. cement No. 4 per 100 gm. clay	4.07	4.09	6.3	6.3	6.7	6.7
11 gm. cement No. 4 per 100 gm .clay	6.14	6.15	9.6	9.6	10.6	10.6
Cement No. 4	62.30	62.75	---	---	---	---
3 gm. cement No. 5 per 100 gm. clay	1.99	1.99	2.9	2.9	3.0	3.0
7 gm. cement No. 5 per 100 gm. clay	4.12	4.13	6.2	6.2	6.6	6.6
11 gm. cement No. 5 per 100 gm. clay	6.16	6.12	9.4	9.3	10.4	10.3
Cement No. 5	63.50	63.92	---	---	---	---
3 gm. cement No. 6 per 100 gm. clay	1.99	1.98	2.9	2.9	3.0	3.0
7 gm. cement No. 6 per 100 gm. clay	4.12	4.19	6.3	6.4	6.7	6.8
11 gm. cement No. 6 per 100 gm. clay	6.20	6.25	9.6	9.6	10.6	10.6
Cement No. 6	63.10	63.60	---	---	---	---
CECIL CLAY	None	None	---	---	---	---
3 gm. cement No. 1 per 100 gm. clay	1.90	1.83	3.0	2.9	3.1	3.0
7 gm. cement No. 1 per 100 gm. clay	4.11	4.08	6.5	6.5	6.9	6.9
11 gm. cement No. 1 per 100 gm. clay	6.23	6.14	9.9	9.7	11.0	10.7
Cement No. 1	62.80	63.20	---	---	---	---
3 gm. cement No. 2 per 100 gm. clay	1.82	1.75	3.0	2.8	3.1	2.9
7 gm. cement No. 2 per 100 gm. clay	3.98	3.90	6.5	6.3	6.9	6.7
11 gm. cement No. 2 per 100 gm. clay	5.97	5.94	9.7	9.6	10.7	10.6
Cement No. 2	61.30	61.78	---	---	---	---
3 gm. cement No. 3 per 100 gm. clay	1.91	1.90	3.0	2.9	3.0	3.0
7 gm. cement No. 3 per 100 gm. clay	4.18	4.17	6.5	6.4	6.9	6.8
11 gm. cement No. 3 per 100 gm. clay	6.28	6.26	9.7	9.7	10.7	10.7
Cement No. 3	64.55	64.78	---	---	---	---
3 gm. cement No. 4 per 100 gm. clay	1.87	1.81	3.0	2.9	3.1	3.0
7 gm. cement No. 4 per 100 gm. clay	4.03	4.03	6.5	6.4	6.9	6.8
11 gm. cement No. 4 per 100 gm. clay	6.00	6.03	9.6	9.6	10.6	10.6
Cement No. 4	62.30	62.75	---	---	---	---
3 gm. cement No. 5 per 100 gm. clay	1.89	1.88	3.0	2.9	3.1	3.0
7 gm. cement No. 5 per 100 gm. clay	4.12	4.06	6.5	6.4	6.9	6.8
11 gm. cement No. 5 per 100 gm. clay	6.21	6.19	9.8	9.7	10.8	10.7
Cement No. 5	63.50	63.92	---	---	---	---
3 gm. cement No. 6 per 100 gm. clay	1.94	1.90	3.1	3.0	3.2	3.1
7 gm. cement No. 6 per 100 gm. clay	4.18	4.09	6.6	6.4	7.1	6.9
11 gm. cement No. 6 per 100 gm. clay	6.12	6.17	9.7	9.7	10.7	10.7
Cement No. 6	63.10	63.60	---	---	---	---

In 4 of the 6 mixed samples listed in table 1, the percentages of cement found by the 2 methods are identical. Of the remaining 2 samples, the Public Roads method gives a result 0.1 percent lower in 1 case and 0.1 percent higher in the other. Of the 36 samples listed in table 2, the 2 methods give identical results on 14 samples. The results obtained by the Public Roads method are 0.1 percent lower on 14 samples, 0.2 percent lower on 4, 0.1 percent higher on 3, and 0.2 percent higher on 1.

These differences are listed in table 3, columns 3 and 6. The average of the algebraic sum of the differences is -0.4 percent. This minus value shows that the Public Roads method tends to give slightly lower values for the cement content of soil-cement mixtures than the method recommended by the subcommittee.

TABLE 3.—*Differences in cement content found by the two methods*

Subcommittee method	Public Roads method	Difference	Subcommittee method	Public Roads method	Difference
Percent	*Percent*		*Percent*	*Percent*	
4.9	4.9	---	2.9	2.9	---
7.0	7.0	---	6.3	6.4	+0.1
8.2	8.1	−0.1	9.6	9.6	---
4.9	4.9	---	3.0	2.9	−.1
6.8	6.8	---	6.5	6.5	
8.2	8.3	+.1	9.9	9.7	−.2
2.9	2.9	---	3.0	2.8	−.2
6.5	6.5	---	6.5	6.3	−.2
9.7	9.6	−.1	9.7	9.6	−.1
2.9	2.9	---	3.0	2.9	−.1
6.4	6.3	−.1	6.5	6.4	−.1
9.4	9.5	+.1	9.7	9.7	---
2.9	2.9	---	3.0	2.9	−.1
6.2	6.4	+.2	6.5	6.4	−.1
9.6	9.5	−.1	9.6	9.6	---
2.8	2.9	+.1	3.0	2.9	−.1
6.3	6.3	---	6.5	6.4	−.1
9.6	9.6	---	9.8	9.7	−.1
2.9	2.9	---	3.1	3.0	−.1
6.2	6.2	---	6.6	6.4	−.2
9.4	9.3	−.1	9.7	9.7	---

SUMMARY

A method for determining the percentage of cement in soil-cement mixtures is described. Data are presented giving the results of the determination of percentage of cement in soil-cement mixtures by this method and by the method recommended by Subcommittee No. 2 of the Project Committee on "Stabilized Roads" of the Highway Research Board as a suitable method of test. In nearly half the samples the percentages of cement found by the two methods are identical. In the remaining samples, the variation is not greater than 0.2 percent. By eliminating the double precipitation of iron, aluminum, and other ions this method offers a saving in time without sacrifice of accuracy.

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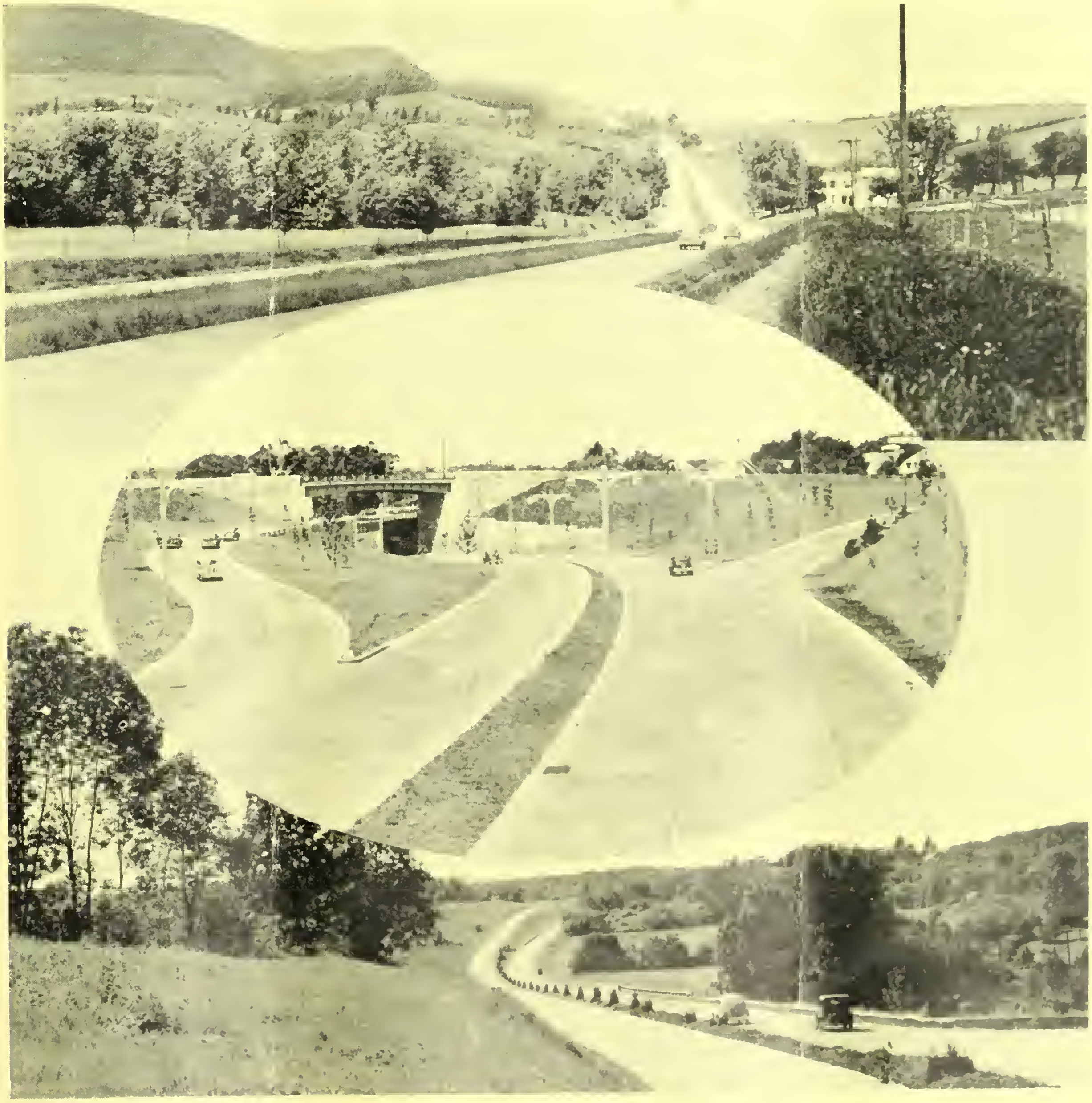

LAND ACQUISITION IS AN IMPORTANT ELEMENT OF MODERN HIGHWAY CONSTRUCTION

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LAND ACQUISITION FOR HIGHWAYS IN NEW YORK STATE

A STUDY OF METHODS OF ACQUIRING LAND FOR HIGHWAYS, PARKWAYS, AND GRADE-CROSSING ELIMINATIONS

Reported by DAVID R. LEVIN, Transportation Economist, Division of Financial and Administrative Research

PUBLIC HIGHWAY authorities in the State of New York are finding that they cannot obtain land for State highway facilities when they need it. When the land is finally acquired, it is found that not enough has been taken, and that too much has been paid for land purchased. An obsolete land acquisition policy threatens to distort the growth of a State highway system which might otherwise enjoy a wholesome development.

To properly implement the creation and modernization of highways and parkways, land acquisition policy must facilitate land assembly at the minimum total cost with a maximum of speed consistent with the preservation of private property rights. To this end, the following are recommended:

(1) A single, efficient method of acquiring lands for highway facilities or for all public purposes should be substituted for the many cumbersome procedures now used.

(2) The right of entry to make locations and surveys ought to be written into the State statutes, as an aid in the construction of public roads.

(3) While the right of entry and possession, granted under the various laws surveyed in this investigation, are perhaps sufficient to expedite the construction of highway facilities, a uniform method granting the right upon notice to the owner of the appropriation, as under the Grade Crossing Elimination and Conservation Laws, is highly desirable.

As this article went to press, two bills pending in the New York Legislature, were enacted into law and became Chapters 194 and 544, Laws of New York, 1944. Under this new legislation, the State Superintendent of Public Works is empowered to finance and acquire lands for rights-of-way on the State system. The method of land acquisition prescribed is the administrative Court of Claims system, substantially the same as that now being employed in the grade-crossing-elimination program, recommended in this article.

This is a forward step in land acquisition practice. Other States may well follow the New York example.

(4) A centralized State land acquisition department should be created in the New York State Division of Highways, provided with adequate records, and staffed with trained personnel selected under a merit system.

(5) The judicial condemnation mechanism, exemplified by the Highway Law, is cumbersome and costly. The method of forcible acquisition should be revised and modeled after the summary and effective administrative Court of Claims procedure, exemplified by the Grade Crossing Elimination Act.

(6) All court acquisition proceedings should be given a genuine preference over all other civil actions in the setting of times of hearing and trial, so that all such actions may be quickly heard and determined.

(7) While the law permits special benefits to property arising out of public improvements to be offset against consequential damages, in actual practice it is rare that benefits of any kind are taken into account in determining just compensation. The sound public policy of offsetting benefits against damages should be applied in practice.

(8) Since the financial incapacity of local units is becoming more and more an obstacle in the provision of adequate State highway facilities, the State should finance land acquisition, particularly where improvements are of little local concern.

(9) The device of marginal land acquisition, sometimes identified as "excess condemnation," can be used effectively in New York State, in its highway, parkway, and grade-crossing-elimination programs. It is used but sparingly today.

THE RIGHT-OF-WAY ACQUISITION PROBLEM

Thirty-seven years ago, the Governor of New York indicated his concern about the public land acquisition practices of that time. "The delays and expense incident to condemnation proceedings have been the occasion of much just criticism. It should be possible to devise a plan by which the appraisal of the value of land taken for public uses can be made by a permanent body with fixed compensation. This would be in the interest of economy and dispatch, and would tend to put an end to the abuses which have too frequently existed in the past."[1] That this condition still exists today in even greater measure is a sad commentary upon the progress of public land acquisition technique since the turn of the century.

Field investigation of the legal machinery and administrative practices in acquiring land for State highways and parkways reveals striking inefficiency and obsolescence. With the establishment of the State highway department, New York, like many other States, fell heir to ancient land acquisition procedures; and with few exceptions, little revision or simplification has been attempted. Possibly the greatest single impediment in the provision of a modernized highway system is the inability of the State to acquire efficiently the lands necessary for highway purposes.[2]

For some time, there has been agitation to reform the mechanism by which New York and its political subdivisions forcibly acquire lands for highway facilities. Inherent obstacles threaten the transport structure built up and maintained in recent years and impede its modernization. This investigation was undertaken to compare and appraise the various practices in New York in the acquisition of land for State highways, for highway-railroad grade-crossing eliminations, and for parkways.[3] It is the purpose of this commentary to outline the criteria of an effective public land acquisition policy and to indicate the extent to which New York practices meet desirable standards.

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[1] From the Governor's message, New York, 1907.

[2] For a discussion of the right-of-way acquisition problem in the United States as a whole, see a pamphlet issued by the Public Roads Administration entitled "Public Land Acquisition for Highway Purposes," by David R. Levin, June 1943.

A SINGLE EFFICIENT LAND ACQUISITION METHOD IS NEEDED

A principal weakness of right-of-way acquisition practice in New York is the variety of methods, voluntary and compulsory, of acquiring lands. Of the many laws which provide for the acquisition of private property for public purposes in the State, but four have been surveyed in this investigation, namely, the Highway Law, the State-wide Grade Crossing Elimination Act, the Syracuse Grade Crossing Elimination Act, and the Conservation Law.[4] Despite the substantial identity of their purposes, each of these laws designates a different method for the acquisition of lands, three of them cumbersome, costly, and time-consuming.

The land acquisition machinery designated in these laws may be roughly classified as either judicial, exemplified by the Highway Law, or administrative, as illustrated by the Grade Crossing Elimination Act. The administrative system is a modern development.

Under the judicial method of public land acquisition, after necessary preliminary steps have been taken, the condemner seeks to purchase the property through negotiation; if the power of eminent domain must be resorted to, the condemner invokes the formal legal machinery by service of a petition upon the condemnee-owner, the court after hearing the evidence determines, with or without a jury, the just compensation for the property, and title passes after the award has been paid the owner. Under the administrative type, after certain preliminaries, the condemner files a plat and description of the property, and after notice to the owner of such action, the appropriation is complete and title to the property vests in the State. If formal offers of the condemner are rejected, the owner must file a claim for the value of his property taken or damaged with the State Court of Claims which makes an award after hearing all the evidence.

While the theoretical legal foundations of public authority for each of the laws investigated can be distinguished, practically, the common objective is the acquisition of private property for a public use or purpose. In the forcible acquisition of rights-of-way for highways, the power of eminent domain is exercised for a public purpose; lands for grade-crossing eliminations are taken under the State police power to protect the public health, safety, and morals; while rights-of-way for parkways are acquired under the power of eminent domain for the preservation of natural resources. In all cases, it is recognized that just compensation for the taking of private property must be made, and that in so doing, due process of law must prevail. A single uniform method, preferably of the administrative type, for the acquisition of all lands for public purposes in New York can be effectively substituted for many present cumbersome procedures.

Entrance to private property to study road location and make surveys is a necessary activity precedent to the construction of roads. It may be that the right of entrance for surveys is implied if not expressly granted by statute, but it is highly desirable that it be specifically indicated in the law.

None of the land acquisition methods surveyed in New York makes specific provision for entry for surveys. New York may well follow the lead of the few States that have granted the right by law, specifying that the entry and survey must be made in a manner compatible with the greatest public benefit and the least private injury, and that highway authorities will become liable for injuries resulting from negligence or malice.

RIGHT OF IMMEDIATE POSSESSION OF LANDS MOST IMPORTANT

The delay and expense experienced in many land acquisitions may be attributed, in large part, to the absence or inadequacy of a legal right to enter and take possession of the desired premises after certain preliminary negotiations have been completed.

While a right of entry and possession is granted under the various laws surveyed in this investigation, it is granted in a different manner and at varying times under each method of land acquisition. Under the Highway Law, a right of entry exists, presumably, as soon as an agreement is executed; in the event condemnation is necessary, the filing of the oaths of commissioners of appraisal vests the title to the lands in the county whereupon a right of entry exists. This is likewise true of acquisitions under the Syracuse Grade Crossing Elimination Act and for the Bronx Parkway Extension. Under the court or referee system, utilized only in Erie County, county agents may enter upon the lands after a copy of the condemnation petition and a map of the parcel have been filed with the county clerk and notice has been given to the owner. The State-wide Grade Crossing Elimination Act, as well as the Conservation Law under which the Eastern State Parkway was established, provide that notice to the owner of the appropriation by the State completes the title, whereupon the State may enter upon the property.

The right of entry and possession thus variously granted is substantially sufficient to expedite the construction of highway facilities, but a uniform method granting the right upon notice to the owner, of the appropriation as under the Grade Crossing Elimination and Conservation Laws, is highly desirable.

CENTRALIZED STATE LAND ACQUISITION ORGANIZATION AN IMMEDIATE SOLUTION

The establishment of an efficient, centralized land acquisition department as an adjunct to the State highway department may well provide the basis for an immediate solution of the right-of-way problem in New York. This is particularly true with reference to the acquisition of lands for State highways, where the local units of government continue to acquire rights-of-way for a highway system which has rendered their land acquisition activities obsolete.

Survey of the present organizations which acquire lands for highways, parkways, and grade-crossing eliminations brings to light a motley array of agencies. The county boards of supervisors, operating through special right-of-way committees, are authorized to acquire lands necessary for State highways; of 26 counties investigated, only two maintain separate, full-time right-of-way organizations, namely, Westchester and Erie Counties.[5] In contrast, all lands required for grade-crossing eliminations (except in New York City, Buffalo, and Syracuse) are acquired by a central State agency, the Grade Crossing Bureau of the Department of Public Works. This bureau has an adequate staff of trained experts, maintains complete records of its transactions, and follows practices that have made it one of the most efficient land acquisition bodies in the State. Midway between the extremes of county acquisition for State highways and State acquisition for grade-crossing eliminations, is the authority to acquire lands for grade-crossing eliminations in the city of Syracuse.

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[3] The study was undertaken in 1940 by the Public Roads Administration under a cooperative agreement with the University of Wisconsin. It was directed by Dr. Henry R. Trumbower, Senior Agricultural Transportation Economist, Public Roads Administration, and Professor of Transportation, University of Wisconsin. H. R. Briggs, A. R. McGinley, and the author supervised the project.

[4] See p. 313 for a complete analysis of these laws.

Authority to establish the Eastern State Parkway is lodged with the five-member Taconic State Park Commission which acquires the necessary lands under a summary administrative method prescribed in the Conservation Law. While the acquisition machinery itself seems adequate, its operation in connection with the Eastern State Parkway has not been effective. Finally, the eight-member Westchester County Park Commission, as agent for the State, acquired the rights-of-way necessary for the Bronx Parkway Extension. While the procedure followed is substantially similar to that used for highway rights-of-way, efficient administration has tended to mitigate the destructive influences of an anachronistic land acquisition mechanism.

These are examples of the land acquisition agencies in the State. The need for extensive revision of the present system, particularly that for State highways, is apparent from an examination of the characteristics of an efficient centralized land acquisition organization and from comparisons with existing practices.

Skilled personnel.—Specially trained and expert personnel are required to cope with the complex right-of-way requirements of modern public highways. Engineers, lawyers, negotiators, appraisers, photographers, and other experts have convincingly demonstrated their efficiency by ultimate savings in total acquisition costs.

A primary weakness found in right-of-way acquisition in New York was the absence of full-time permanent agents qualified in the tasks they were to perform. Isolated exceptions were found. Erie and Westchester were the only counties with fairly adequate personnel for land acquisition. On the other hand, early and consistent provision has been made for trained personnel with specialized skills, employed on a full-time basis, in land acquisition for the State-wide grade-crossing-elimination program. In contrast with elected and appointed county personnel, every individual associated with grade-crossing land assembly is a civil service selectee. It is doubtful whether the volume of work which the Syracuse Grade Crossing Commission undertakes is sufficient to justify a more specialized organization than exists at present. Results of a survey of both the Taconic State Park Commission and the Westchester County Park Commission as agents for the State points to the urgent need for an over-all State organization, similar to the Grade Crossing Bureau, to facilitate the acquisition of lands for State highways, grade-crossing eliminations, parkways, or other road facilities.

Adequate records.—Public land acquisition is a business enterprise and adequate records of all transactions should be maintained. Every legal document, letter, record, and written statement concerning a given highway development should be retained in a central office, classified according to route number and subclassified by consecutive parcels or alphabetically by the name of the grantor. These records would include title searches, reports on title examinations, appraisals, special reports, and all correspondence. Cost ledgers should be maintained indicating for every parcel, *inter alia*, the following items: Land costs per acre and for the parcel; damages, segregated as to improvement and consequential damages and the nature of each of these; consequential benefits, if any; incidental costs, such as title search fees, appraisal fees, and condemnation costs in detail.[6]

Of the five agencies surveyed, the records of the State Grade Crossing Bureau were by far the best. County records concerning land acquisition activities are generally absent altogether and such as do exist are amazingly scanty and inadequate; the methods used are highly diverse because of the many different units involved. Of the 26 counties investigated, the best records were found in Erie, Westchester, Columbia, and Schenectady Counties, but even in these places there is much that can be improved. The records concerning the Eastern State Parkway, the Bronx Parkway Extension, and Syracuse grade-crossing eliminations may be characterized as mediocre.

A good illustration of the type of record keeping desirable in right-of-way acquisition may be found in the Grade Crossing Bureau. While the system used is not perfect in every respect, it does include the more important features. The centralized records consist of a series of folders containing all information pertaining to each project, and to each parcel within such project, filed under a special Public Service Commission number. Each folder contains a map of the entire project, plat maps for each parcel, engineers' estimates of land and damage costs, reports of physical inspection of the property, copies of formal notices, photographs of the properties, copies of agreements, Court of Claim proceedings, as well as correspondence and memoranda concerning all transactions. Included also are memoranda indicating the basis of settlement, in terms of land costs, and the various elements of damages. Out-of-pocket expenditures for incidental services, such as witness fees, etc., were shown only to a limited extent.

If it should be deemed inexpedient for the State to assume control of land acquisition for State highways, then there is urgent and immediate need for those units doing the work to adopt an adequate system of records. Standard forms should be evolved after some study and every county in the State should be required to use them with uniform interpretation and to maintain records in a single office at the county seat.

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[5] This county procedure is prescribed by the Constitution of the State of New York in the following terms in art. 1, sec. 7: "When private property shall be taken for any public use, the compensation to be made therefor, when such compensation is not made by the State, shall be ascertained by a jury, or by the Supreme Court with or without jury, but not with a referee, or, in proceedings affecting property located within the city of New York and to be acquired by the city of New York, by a term of said court to consist of one or more justices thereof without a jury, or by not less than three commissioners appointed by a court of record, as shall be prescribed by law." In view of the travail generally involved in securing constitutional amendments and the multiplicity of administrative units involved, the desirability of a centralized State public land acquisition organization is compelling indeed.

[6] The New York Supreme Court has ruled that when property is acquired for a public purpose, the following facts should be indicated. (1) The elements of cost divided among land, buildings, consequential damages, and consequential benefits. (2) the manner in which each of these items is affected, (3) the amount of damage or cost allowed on each item, and (4) the rule by which such values were determined. See *Matter of Board of Supervisors*, 140 Misc. (N. Y.), 894.

Prenegotiation appraisals.—A prenegotiation appraisal is a scientific determination of the value of a given parcel, in terms of land, improvements, and consequential damages. Its purpose is to provide the basis for an intelligent and fair settlement with the owner. A condemnation appraisal is such a determination after negotiations have broken down and court proceedings are under way. Its purpose is to justify the State's or the county's assertion of value of the parcel.

Examination of the present appraisal practices in New York brings much to light that can be improved. Only three of the many counties studied make prenegotiation appraisals, namely, Westchester, Erie, and Schenectady. The efficiency of this system in Erie County is reflected in the close approximation of the final settlement to the appraised values, the variation being less than ten percent. In the remaining counties, only a condemnation appraisal is made, and only when it is essential to the prosecution of eminent domain proceedings. Under the procedure generally used, the owner is likely to be well paid for his efforts in bargaining.

The approach in determining property values in a majority of counties is basically unsound. Widespread absence of the prenegotiation appraisal, explains why some property owners are recalcitrant and compel the county to condemn their properties. Force of logic indicates that court proceedings, necessitating a condemnation appraisal, may be the only basis upon which an intelligent and fair award can be made. It is not at all unlikely that the number of condemnation suits could be materially reduced through the more extensive as well as intensive utilization of the prenegotiation appraisal. No brief is held for either condemner or condemnee and it is true, no doubt, that some property owners demand exorbitant prices. But it seems that county authorities are "putting the cart before the horse" when they attempt to purchase property without first making an impartial and scientific appraisal, waiting until they get into court to ascertain what they ought to pay the owner for his property.

Appraisals are particularly important in grade-crossing-elimination negotiations. The nature of the structures required results in high consequential damages due to severance and changes in grade. The Grade Crossing Bureau staff includes full-time, expert appraisers and claims adjusters who furnish detailed appraisals of every tract of land to be acquired for elimination purposes. The wisdom and economy of this practice is reflected in the close agreement of settlements with the appraised values and the relatively few contested cases.

The appraisal techniques of the Taconic State Park Commission and the Westchester County Park Commission are mediocre, although prenegotiation appraisals are made to a considerable extent. Settlements of the Syracuse Grade Crossing Commission are subject to the approval of the corporation counsel and the comptroller.

Base or standard values.—The use of base or standard values in the appraisal process promotes a fair and consistent determination of land and property awards. Base values are purchase prices set by public authorities for all bare or "raw" land and for certain improvements and betterments such as fencing and trees susceptible of unit-of-value measurement. Except in special cases, authorities will deviate but little from these values in bargaining with owners of property. To illustrate, the right-of-way department may determine that agricultural pasture lands in Albany County are worth $75 an acre and that ordinary fencing is worth $1 a rod and bargain accordingly with property owners when land is desired for highway purposes.

The device has been used but sparingly in New York despite its potential usefulness, due in part to the multiplicity of local units which participate in public land acquisition, particularly for State highways, and in part to the anachronistic procedure under which such units function. The Taconic State Park Commission has used the device on the Eastern State Parkway with considerable success. Classification is based upon topography and land use, obtained through field inspection by the chief forester and the commission appraisers. Distinction is made between rolling meadow lands, rocky hillsides, orchard lands, swamps, and other types.

Photographs.—Photographs of parcels of property are often valuable in negotiations with the owner and in court proceedings, especially where the condemner takes immediate possession of the premises. What a person sees is many times more effective than what he hears.[7]

With the exception of the State Grade Crossing Bureau and Schenectady County, the land acquisition agencies investigated rarely use photographs. It is now standard practice with both organizations mentioned to take photographs of every parcel acquired, before, during, and after construction of improvements. The photographs are made part of the land acquisition records. Both organizations consider the device indispensable.

Research.—In recent years, highway authorities have recognized the need for evidence to substantiate allegations of inadequacy and obsolescence of existing facilities, of exorbitant land acquisition costs, and of legal and administrative incapacity to cope with the expanding requirements of modern motor transport. Research of this nature is entirely lacking in New York. A division of a centralized right-of-way department might well devote its energies to research along these legal and economic lines. It has been alleged that many condemnation cases have been "lost" because counsel lacked evidence to establish their contentions. There is urgent need for assemblage and interpretation of all the facts on right-of-way practices and procedure, in the same manner that the State-wide highway planning surveys, under guidance of the Public Roads Administration, have undertaken the task with respect to the highway system and its use.

TITLE SEARCHES AND EXAMINATIONS ARE EXPENSIVE AND TIME-CONSUMING

Because of the many owners of parcels of land and the frequent transfers of such ownership, the need for an adequate system of land registration is immediate and compelling. Examination of the systems of title searching and title examination in the transfer of property in New York indicates that the procedures are expensive and time-consuming.

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[7] The National Association of Real Estate Boards has found that 87 percent of a sale was effected through the eye, 7 percent through the ear, and the remainder through the other senses.

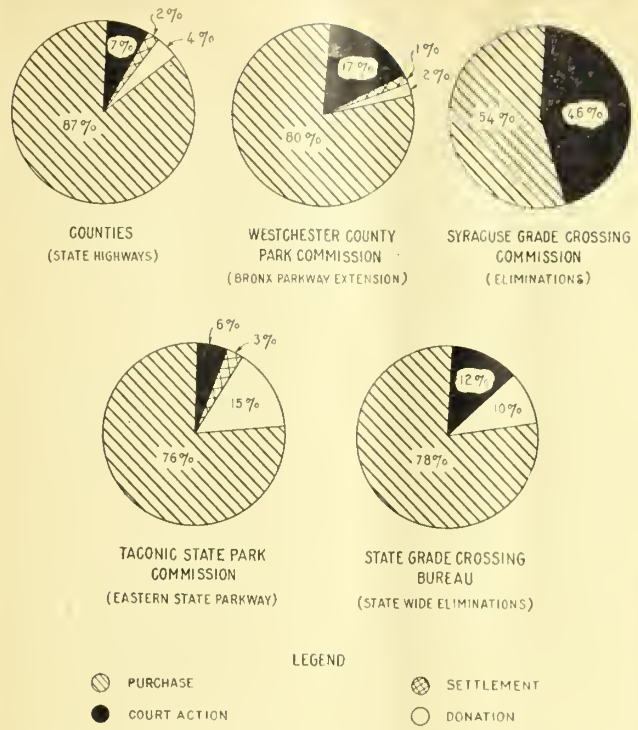

Figure 1.—Percentage Distribution of Total Area by Manner of Acquisition for Five Agencies Surveyed New York State, 1940.

Under the Torrens system of land registration, all deeds and documents affecting property are registered and a single certificate indicating who the owner is and what encumbrances exist on a given piece of property is readily available. This contributes to the easy marketability of title and greatly diminishes the delays and expense of title abstract extensions and title searches incident to ministerial or judicial transfers of land. The need for expensive and elaborate title searches in connection with the acquisition of lands for public improvements is obviated.

While New York has the necessary enabling legislation for land title registration, the device is utilized only rarely. A recent study has disclosed that approximately only one-tenth of one percent of real estate title proof took place under the land title registration system.[8] Instead, a title search is made at great expense and with unreasonable delay. In the acquisition of lands for State highways, title search costs ranged from $3 to $40 per parcel, exclusive of examination expenses, in the counties surveyed. Even under the otherwise efficient grade-crossing-elimination program, such costs ranged from $9 to $293 per parcel, the average being approximately $45. The spread for Syracuse grade-crossing eliminations was from $7.50 to $90, with the average between $40 to $50 per parcel. Title search fees for parkway land acquisitions were high, ranging from $5 to $410 per parcel for the Eastern State Parkway, and from $10 to $500 for the Bronx Parkway Extension. In the aggregate, title search costs constituted the largest single item of incidental expense. The magnitude of these expenditures warrants a careful consideration of the merits of the Torrens system of land registration.

LANDS ARE ACQUIRED BY DONATION, PURCHASE, AND CONDEMNATION

Lands for highways, parkways, and grade-crossing eliminations in New York have been acquired variously through donation, purchase, settlement,[9] and condemnation. Figure 1 indicates the relative percentages of the total area by manner of acquisition for State highways, State-wide and Syracuse grade-crossing eliminations, the Eastern State Parkway, and the Bronx Parkway Extension.

Approximately 7 percent of the land acquired for State highways was condemned, while almost half of the area required for Syracuse grade-crossing eliminations was obtained through court action. Approximately 12 percent of lands required for State-wide grade-crossing eliminations was obtained through proceedings in the Court of Claims. While these data would seem, to place State highway acquisition in the most favorable light, these bare statistical facts are meaningless unless interpreted in light of general acquisition practices, such as the appraisal, negotiation, and condemnation policy. The efficiency of the acquisition machinery must be determined in the light of over-all cost and speed of acquisition.

COURT PROCEEDINGS ARE CUMBERSOME AND INEFFICIENT

Lands for highway facilities that cannot be obtained through negotiation and purchase must be acquired forcibly through court proceedings. Lands for State highways, Syracuse grade-crossing eliminations, and the Bronx Parkway Extension were acquired by the judicial condemnation process, while lands for State-wide grade-crossing eliminations and the Eastern State Parkway were taken forcibly by administrative appropriation and settlement in the Court of Claims.[10] The acquisition process as a whole is cumbersome, costly, and inefficient, particularly the judicial condemnation mechanism.

To illustrate, the Westchester Park Commission desired about five acres of Mrs. X's farm for the Bronx Parkway Extension. Three appraisers valued the property at $1,742, $1,314, and $2,080, respectively, all of which were in excess of the current market values of similar property. The commission offered Mrs. X $2,570 for the 5.1 acres of ordinary unimproved farm land. Little damage would be done the remaining land by parkway improvement. Two years later, the attorneys for Mrs. X protested to the Governor concerning the unnecessary delay and expense, indicating that the condemnation commission had held 40 meetings and had only met and adjourned at half of the meetings. Each of the members received $25 per day for each of these sessions. Finally, after almost four years of negotiation and litigation, Mrs. X received an award of $8,368, and the total additional out-of-pocket expense of the condemnation proceedings to the State was $8,964.

A drastic revision of procedure seems indicated, and probably should be modeled after the administrative Court of Claims procedure. Redesign of the court acquisition machinery should take into consideration the following elements.

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[8] See "Systems of Land Title Examination: An Appraisal" by H. Russell and D. A. Bridewell, The Journal of Land and Public Utility Economics, May 1938, p. 134, et seq.

[9] Lands are acquired by settlement when condemnation proceedings are initiated for a given parcel, but the owner comes to terms and settles "out of court."

[10] For the distinction between the judicial and administrative processes, see discussion, p. 302.

Court tribunals.- The number and types of tribunals designated to hear condemnation suits are many and diverse. The five different land acquisition laws surveyed designate five different court tribunals to try cases of forcible acquisition. The highway law lodges authority in the county court, the supreme court, and the three-member commission of appraisal in all counties except Erie County, which may utilize a Supreme Court Justice or an official referee, at its option. All court proceedings arising under the Grade Crossing Elimination Act are tried in the State Court of Claims, while the Syracuse Grade Crossing Elimination Act designates the county court and the commission of appraisal, as does the Eastern State Parkway legislation. Claims arising out of land acquisition for the Bronx Parkway Extension were tried in the Court of Claims. Neither the character of the condemner nor the purpose for which property is taken justifies such diversity.

There is a very positive correlation between the purchase and court acquisition policies followed by a given agency. The Grade Crossing Bureau has established an enviable record in its purchase policy, in the 16 years of its existence. Examination of a reliable sample of 3,000 parcels acquired for grade-crossing eliminations in all portions of the State indicates that only one-half of one percent was acquired through court proceedings.

A number of explanations may account for the ability of the bureau to acquire practically all of the necessary lands through voluntary negotiation. Purchases are based upon policies which other land acquisition organizations in the State may well adopt. A bona fide attempt is made by the State to settle each case on the basis of fair values ascertained in a scientific manner. While negotiating practices are flexible, the policy of rather rigid adherence to appraised values prevails. Such a practice, consistently followed, soon becomes well known among the property owners affected by grade-crossing eliminations as well as the bar, with the concomitant realization that nothing can be gained by rejecting reasonable offers.

This is in contrast with the current policy of many counties in acquiring lands for State highways. In dealing with counties, property owners are likely to be well paid for their efforts in bargaining or in condemnation proceedings. The Grade Crossing Bureau is not thwarted by dilatory practice. It has the power, after summary preliminaries, of immediate entry and possession upon the lands of recalcitrant owners. The burden of invoking legal remedies for the recovery of compensation in excess of the State's concept of fair value devolves upon the property owner rather than upon the condemner as in county acquisition of State highway rights-of-way.

Perhaps one of the best explanations of the relatively successful purchase and court policy of the grade-crossing organization is found in the nature of the Court of Claims. Under the law, property owners who file claims for damages with that tribunal do so at their own risk and expense; irrespective of the outcome of the case, they are not reimbursed for their court costs, disbursements, and attorney and witness fees. There is little incentive, therefore, for owners to resort to court proceedings.

Preference of land acquisition cases on the court calendar.—Because public highway improvements are many and sorely needed and because court dockets are congested, condemnation suits ought to be preferred on the court calendar. Except in isolated cases, little provision is made in the State for preferences in court cases. While Erie County cases and suits arising out of the Eastern State Parkway improvement enjoyed a theoretical preference in court, little effort was exerted to make such a preference effective. It is suggested that the present land acquisition laws be revised, to the end that all forcible acquisition proceedings be given preference over all other civil actions in setting times for hearing and trial. Means should be devised to make such a preference effective in practice.

Group court proceedings.—Group condemnation—the prosecution of eminent domain proceedings against a number of separate parcels or owners in a single suit—ought to be sanctioned by law and used effectively in the acquisition of lands for highway facilities. The device is used in New York only to a limited extent. Warren, Westchester, and Schenectady Counties use group condemnation effectively in the acquisition of lands for State highways, while the other land-acquiring agencies surveyed utilize the technique but sparingly. Its widespread adoption is recommended.

Abandonment of condemnation proceedings.—When the need arises, the condemner of lands should be permitted to abandon condemnation proceedings except where the property is deemed already to have been taken. It may develop that the particular property in question is no longer desired for the improvement, or the award may be excessive. The laws of New York make no provision for abandonment.

Friendly, noncontested, summary court proceedings.—Friendly, noncontested, summary court proceedings should be sanctioned in unusual cases and to cure defective titles to property acquired for public purposes. An illustration of the need for such a device was found in this survey. Warren County attempted to purchase from Syracuse University certain lands for a highway improvement, but difficulties arose because the land had been donated originally to the university for educational purposes and now was to be dedicated to highway uses. Condemnation proceedings had to be instituted to solve the problem.

COSTS OF COURT ACTION HIGH

It is most difficult to make fair comparisons of the efficiency of the forcible acquisition procedure of one system with that of another, or of one device with another on the basis of costs alone. Other factors are present, the effect of which cannot be separated statistically. For example, there may be fewer court cases under one procedure than under another; yet it cannot be said, per se, that the method generating the fewer contested cases is the more efficient. The competence of a system of land acquisition is a function not only of the number of parcels condemned, but also of the nature of the individual acquisitions, the final offers and awards for all purchased parcels as compared with the true market values, the normal purchase policy, the length of negotiation, land uses, population density, and a host of other influences. Any analysis of court cases which takes these variables into account will indicate that the resultant sample is much too small to be reliable.

Any detailed analysis of cost statistics is to be treated with caution but a few general conclusions are apparent from this survey. While the expense of court action in the acquisition of lands for highway facilities is uni-

TABLE 1.—*Summary of cost elements for parcels acquired by court action under various methods of compulsory acquisition*

Item	Number of parcels	Area in square yards	Total cost		Property cost		Total cost of court action			Incidental cost	
			Amount	Percent	Amount	Percent	Amount	Cost per parcel	Percent	Amount	Percent
Condemnation:											
State highways	70	471,204	$172,810	100	$119,608	70	$52,624	$752	30	[1] $578	
Syracuse grade-crossing elimination	23	4,371	86,257	100	60,448	70	23,457	1,020	27	2,352	3
Bronx Parkway Extension	6	336,350	138,098	100	84,421	61	52,431	8,739	38	1,213	1
Court of Claims appropriation:											
State-wide grade-crossing elimination	19	87,105	44,253	100	37,936	86	5,815	306	13	502	1
Eastern State Parkway	8	1,193,108	95,814	100	79,325	83	15,741	1,968	16	748	1

[1] This includes $488 incidental cost and $90 taxes.

TABLE 2.—*Comparison of cost elements of court proceedings under various methods of compulsory acquisition*

Item	Number of parcels	Total condemnation costs		Commissioners' fees		Attorney's costs		Interest		Other court costs	
		Total	Cost per parcel	Total	Cost per parcel	Total	Cost per parcel	Total	Cost per parcel	Total	Cost per parcel
Total	126	$150,071	$1,191	$47,573	$378	$15,111	$120	$42,088	$334	$45,299	$359
Condemnation:											
State highways	70	52,624	752	22,012	314	5,735	82	7,168	102	17,709	253
Syracuse grade crossings	23	23,457	1,020	12,261	533	4,760	207	1,566	68	4,870	212
Bronx Parkway Extension	6	52,434	8,739	13,300	2,217	4,616	769	17,186	2,864	17,332	2,889
Court of Claims appropriation:											
State-wide grade crossings	19	5,815	306					3,927	207	1,888	99
Eastern State Parkway	8	15,741	1,968					12,241	1,530	3,500	438

[1] These costs include appraiser's fees, witness fees, stenographer's fees, and other miscellaneous court costs.

formly high, table 1 indicates that such expense constitutes a much greater portion of aggregate costs in the judicial condemnation of lands for State highways, for Syracuse grade-crossing eliminations, and for the Bronx Parkway Extension than in the administrative court acquisition of lands for State-wide grade-crossing eliminations and for the Eastern State Parkway.

The percentage in judical condemnation is approximately twice that in administrative court acquisition. The total cost of court action averaged only $306 per parcel for State-wide grade-crossing eliminations, while such costs averaged $8,739 per parcel for the Bronx Parkway Extension, an alarming expense. In many cases, the court expense exceeds the property award itself; and the greater the award, the more serious this situation becomes.

It is not surprising that the counties and acquisition organizations are often "intimidated" into making unreasonable purchase settlements rather than resorting to court proceedings. A typical illustration of inflated property values was found in the records of the Bronx Parkway Extension. The Westchester County Park Commission had appraised a small property at $972 and had offered its owner $1,600, but $6,750 was demanded. Rather than proceed with condemnation, the owner was allowed $6,000.

Court expenses are made up of commissioners' fees, witness and appraiser fees, court costs and disbursements, attorney's fees reimbursed the property owner, interest charges, and miscellaneous items. A comparative analysis of these costs appears in table 2. Compensation costs of commissioners of appraisal in judicial condemnation proceedings have been a weakness of long standing in the land acquisition machinery for State highways, Syracuse grade-crossing eliminations, and the Bronx Parkway Extension. They constituted the largest single item of court expense, ranging from $314 per parcel for State highways, to $2,217 per parcel for the Bronx Parkway Extension. Full-day sessions are not required of commissioners of appraisal and they have every inducement to prolong a contest as long as possible.

For example, the record of a typical State highway condemnation case reveals that out of 16 sessions of the commission, six lasted half an hour, eight a half day, and only two all day. For one simple condemnation case on the Bronx Parkway Extension, the commission held 56 hearings at a cost of $4,200 in commissioners' fees. In another instance on the same improvement, 40 sessions were held, half of which consisted of meeting and adjourning. Many other illustrations could be given.

Other items of considerable magnitude are court costs and disbursements and attorney fees reimbursed the property owner if he is the prevailing party in the condemnation proceedings, i. e., if the court award is in excess of the final offer by the county. Such costs are paid by the condemner to the extent of $65 plus an additional 5 percent of the award. These costs per parcel aggregated $82 for State highway condemnations, $207 for Syracuse grade-crossing eliminations, and $769 for the Bronx Parkway Extension.

In contrast with the judicial condemnation procedure commissioners' and attorney's fees are not permitted under the administrative appropriation system of State-wide grade-crossing eliminations and the Eastern State Parkway. While it is true that these out-of-pocket expenditures are offset in part at least, by that portion of the salaries of the Court of Claims personnel attributable to claim cases, these costs are not large; the characteristic efficiency of that body prevails in land acquisition litigation as in other contests.

Aside from the immeasurable losses from delay in use of needed improvements, unwarranted delays inherent in some of the land acquisition systems result in tangible costs in the form of interest charges. These costs ranged from $68 per parcel for Syracuse grade-crossing eliminations to $2,864 per parcel for the Bronx Parkway Extension. The magnitude of this item alone is sufficient to indict some of the present systems of land acquisition.

Miscellaneous court costs, including appraiser and witness fees, stenographic costs, publication expenses, service of process fees, and related items, ranged from $99 per parcel for State-wide grade-crossing eliminations to $2,889 per parcel for the Bronx Parkway Extension. Aside from these substantial expenditures, the services of many salaried State and county employees are required in contested court cases, such as State attorneys, county attorneys, highway department officials, and their assistants.[11] The general departmental overhead expenses should not be overlooked. Out-of-pocket condemnation costs may range from $500 to $1,500.[12] The prospect of bearing these costs as well as the probability of an unfavorable award has prompted many acquisition organizations in the State to shun forcible court action.

MANY COURT AWARDS CONSIDERED EXCESSIVE

Study of court cases showed that most court awards are excessive. Facts were available concerning condemnation awards for 23 State highway parcels. Total awards were more than double the total of appraised values. Court awards for six parcels condemned for the Bronx Parkway Extension were more that two and one-half times the total appraised values. Final awards for 19 parcels for Syracuse grade-crossing eliminations exceed the appraised values by 45.6 percent. This is the lowest figure of any land acquisition organization surveyed.[13] But consideration of the fact that approximately half of all acquisitions are condemned causes the apparent superiority to vanish.

Even final awards given in the State Court of Claims are substantially in excess of the appraised values. Such awards for 12 parcels for State-wide grade-crossing eliminations exceeded the appraised values by 83.6 percent, while total awards for 8 Eastern State Parkway parcels contested in the Court of Claims were substantially more than double the total of appraised values.

Bare statistical facts, however, should not serve as the sole criteria in determining whether a system of acquisition is efficient or awards excessive. Other factors, such as the tendency toward price inflation in acquirement by voluntary purchase, the appraisal policy, the relative frequency of forcible acquisition, as well as other influences serve to condition the over-all effectiveness of a land acquisition technique. It is significant that 91 parcels were purchased for the Eastern State Parkway at an average cost of only $35 per acre, the damage item being almost negligible. There was little tendency toward price inflation in these purchases. In acquiring State highway parcels in Columbia County, land costs and improvement damages were appraised at substantially the amounts finally awarded, but appraised consequential damages were far less than final awards by the condemnation commission. This was due to a deliberate policy of Columbia County to appraise consequential damage items at a low figure.

In view of the nature of condemnation proceedings and the evidence presented at hearings, condemnation commissions, courts, and juries are sometimes helpless to prevent excessive awards being granted. Too often, when confronted with highly divergent "expert" appraisals by the county and the property owners, commissioners, courts, and jury panels will strike an average between extremes and make an award on that basis.[14] There is something organically wrong with a system that generates such inequitable results.

METHODS OF REDUCING CONDEMNATION COSTS NOT A DIFFICULT PROBLEM

Assuming that it is desirable to retain the framework of the existing land acquisition systems in New York, there are certain devices that may be utilized to reduce condemnation costs. These include the referee system, group condemnation, and prenegotiation appraisals.

Referee system.—Erie County avoids some of the condemnation costs common to other counties in the acquisition of rights-of-way for State highways by utilizing a referee [15] as a special agent of the court instead of the commissioner of appraisal system. While this device is no guarantee against excessive awards, it avoids commissioners' fees which would considerably exceed that portion of the referee's salary chargeable to condemnation cases. Special stenographic fees are avoided by the substitution of a regular court reporter. Since eminent domain proceedings are given preference on the referee's calendar and effort is made to expedite adjudication, interest payments are at a minimum.

Group condemnation.—Group condemnation, as practiced in Warren and other counties, is the prosecution of eminent domain proceedings in a single, consolidated court action, without jeopardizing private property rights. Costs of separate condemnation commissions, repetitive testimony of experts, and other costs are avoided. Full-day sessions are required of commissioners and experts and it has been demonstrated that several cases can be disposed of in a few days. In most counties a separate condemnation commission is appointed for each proceeding with attendant expenses for commissioners and witnesses. There is considerable duplication of evidence in acquiring parcels. Since the parcels on a given highway development are generally in the same locales, many of the evidences of value apply to all in common. Group proceedings are likely to be completed much sooner than a number of separate actions.

Prenegotiation appraisals.—The number of condemnation suits may be materially reduced through prenegotiation appraisals. An efficient acquisition system must be predicated upon an impartial and scientific analysis of fair market values. It is a sad commentary upon present New York practice that the majority of counties wait until they get into the courts to ascertain what they ought to pay an owner for his property.

SPECIAL BENEFITS SHOULD BE RECOGNIZED

Special benefits accruing to property as a result of the improvement for which a portion of the property is taken ought to be offset against damages to the remainder. In New York, such benefits are permitted to be offset only against consequential damages to the remaining portions of the land. The value of benefits cannot be deducted from the value of property taken for the improvement. It was rare that benefits of any kind were taken into account in determination of compensation for acquisitions surveyed in this investigation. While it is true that benefits are sometimes speculative in nature and difficult of precise measurement, it is good public policy to permit their deduction, for it is not just to permit a property owner adjacent to an improvement to reap special benefits from the expenditure of public funds which others cannot share. The owner should be placed in the same position after the forcible acquisition that he enjoyed before his property was taken; in other words, he must be made "whole" as the courts have commanded, but that is all. His position need not be improved, although too often it is.

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[11] No attempt has been made in this investigation to allocate fixed salaries or office overhead to systems of land acquisition or parcels of land acquired.

[12] The average out-of-pocket court expense for the 126 parcels acquired under all systems surveyed averaged $1,191 per parcel. See table 2.

[13] Assuming the sample is reliable.

[14] In making its award, the condemnation commission is not restricted to any species of evidence, but may act upon its own judgment and upon any information which is acquired from inspection of the property, as well as on evidence produced at the hearing. Evidence of experts is not binding, but is advisory only. See *City of Rochester* v. *Casey, Eagan, Falk, et al.*, 234 App. Div. 583 (1932).

[15] Formerly a justice of the supreme court.

FINANCING ACCOMPLISHED BY VARIOUS MEANS

Rights-of-way for State highways must be acquired by counties at their own expense, as directed by State statutes. This survey indicates that some counties have balked at financing land acquisition for improvements designed largely for through traffic, and of little local concern. The obvious financial incapacity of local units is becoming more and more an obstacle to the provision of adequate highway facilities.

All costs of acquiring lands for the Eastern State Parkway as well as the Bronx Parkway Extension are financed by the State through the Taconic State Park Commission and the Westchester County Park Commission, respectively. Grade-crossing eliminations are financed by Federal, State, local, and railroad contributions, in varying proportions.[16]

PUBLIC REPLACEMENT POLICY DESIRABLE

The device of public replacements, despite its potential usefulness in reducing damage costs, is utilized but sparingly in New York. Cattle passes can be built to eliminate damages for severance or change of grade, fences can be replaced by highway department forces to avoid damage payments, and driveways can often be graded as part of the construction contract. If the work is of such a nature that the highway department forces cannot do it economically, contracts can be let after competitive bidding, with resulting savings in costs. Dutchess and Westchester are practically the only counties employing this device, even to a limited extent. Officials in one county alleged that a public replacement policy has not been followed because the county is not authorized by law to use county labor to make highway improvement more efficient.

A vigorous policy of public replacement, particularly of fencing and driveways, has greatly reduced damage payments on the Eastern State Parkway. Refencing was facilitated by a combination of CCC labor and use of lumber from trees felled in making the improvement. Driveways and other improvements damaged were repaired by the parkway construction contractors in the regular course of their activities.

ANALYSIS OF RIGHT-OF-WAY COSTS IMPORTANT

Many factors influence land acquisition costs. The nature of the property acquired and its present and potential uses have a decided effect upon its value. The extent of present highway development will serve to condition the attitude of property owners toward a proposed improvement. The character of the land acquisition organization and its policies have a remarkable influence upon the efficiency of the whole acquisition process, which ultimately is reflected in over-all acquisition costs. A vigorous policy with regard to public replacement of fences and small structures, prenegotiation appraisals, base values, trained personnel and condemnation is vastly superior to the hit-or-miss practices of an unorganized approach.

The relative bargaining abilities of the buyer and seller are not without their influence. This investigation indicates that most owners were well paid for bargaining efforts, particularly those from whom lands were taken for State highways, the Syracuse grade-crossing eliminations and the Bronx Parkway Extension. They obtained greater awards than might have been obtained in dealing with an organization better equipped.

Factors such as population density, the business cycle, real estate market values, special benefits to land as a result of the highway improvement, number and kind of structures and improvements, the extent of court action, and many related influences also determine the magnitude of over-all acquisition cost.

The complexity of these economic pressures is such as to make their respective influences statistically indeterminate. Available samples for analysis of the effect of any one influence are far too few for the purpose. Only a general analysis will be made.

Over-all acquisition costs seem to be a function of the purpose for which lands are acquired and of the acquisition organization. Rights-of-way for State highways are acquired at reasonable cost by some counties while other counties pay seemingly exorbitant prices. Acquisition costs for the Eastern State Parkway are relatively low, with the exception of parcels appropriated by court action. On the other hand, the Bronx Parkway Extension entailed comparatively high costs, due in part to proximity to metropolitan centers, and in part to the organically inefficient land acquisition machinery. State-wide grade-crossing elimination costs are high, because those eliminations most desirable are generally in or adjacent to highly developed areas, and because the very nature of grade-crossing eliminations, involving as they do changes in grade, generates severe consequential damages and often results in irregularly shaped remnants of land having little utility. This is likewise true of Syracuse grade-crossing eliminations, with the added corollary that property costs are generally somewhat greater because of urban influences and more intensive utilization.

Property costs.—Good indices of land values for comparison with highway right-of-way costs are lacking, but there is substantial evidence to indicate that prices paid are considerably in excess of market values. This may be accounted for by the fact that right-of-way costs are not free market values. The purchaser is known in advance and he cannot readily substitute one property for another. When a highway is projected, every parcel needed, with minor exceptions, must be acquired, and neither the purchaser nor seller is free in the sense of being a willing buyer and a willing seller. Highway authorities require designated lands and if government cannot purchase such lands through negotiation, it will resort to its power of eminent domain.

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[16] See p. 313 for a complete discussion of the matter.

Assuming the property is taken for a public purpose, the owner must eventually part with the land, and if he will not negotiate for its sale, condemnation will force his hand. In other words, in public land assembly for highways, as for other public uses, peculiar conditions govern price, circumstances which do not normally prevail on the open market. These special conditions in the aggregate are reflected in what is sometimes labeled plottage or assemblage costs.

The announced policy of at least two counties in New York is to pay more than market values for property required for State highways. One county official indicated that his county ascertained the market value of the property and then offered the owner double that amount. Another county applied this same technique in acquiring smaller tracts of land, but offered 25 percent above the fair market value for properties worth over $500. Yet these two counties were among those having the lowest land costs of those investigated.

Tables 3 and 4 indicate the components of property costs of parcels purchased by the five land acquisition agencies surveyed in New York. Damage payments are the largest element of cost in State highway and grade-crossing-elimination acquisitions while such payments are negligible in the acquisition for the two parkways. The significance of these facts is dealt with in the paragraphs which follow.

Land and damage costs.—The nature of the improvement for which land is acquired has served to condition the magnitude and kind of land and damage costs. State highway development, for the most part, consisted of widening existing roads with relocations here and there. Widening necessitated the acquisition of long, rectangular strips of land adjacent to the existing highways rather than large contiguous tracts of land. Consequently, only about one-quarter of the total property costs is ascribed to land, while three-quarters constitutes damage payments. On the other hand, because they were so planned, both the Eastern State Parkway and sections of the Bronx Parkway Extension traverse largely undeveloped lands. On the Eastern State Parkway, for the 91 parcels purchased, land costs accounted for 92 percent of the aggregate costs, averaging approximately $35 per acre; on the Bronx Parkway Extension, land costs for its 91 purchased parcels amounted to 82 percent of the total costs, averaging $880 per acre. An analysis of the elements of damage payments is indicated in table 5.

TABLE 3.—*Percentage relationship of elements of property cost of purchased parcels for five agencies surveyed*

Kind of development	Property cost	Land cost	Damage cost
	Percent	*Percent*	*Percent*
State highways	100	24	76
Grade-crossing eliminations:			
State-wide	100	29	71
Syracuse	100	20	80
Parkways:			
Eastern State Parkway	100	92	8
Bronx Parkway Extension	100	82	18

Incidental costs.—These fees are those other than land, damage, and condemnation costs arising out of land acquisition activities; they include appraisers' fees, title search and examination fees, recording costs, and so on. This survey is limited in its treatment of incidental costs because only out-of-pocket expenditures were considered; no attempt was made to allocate the salaries of individuals concerned with land acquisition functions. Accordingly, incidental fees averaging approximately $25 per parcel for State highway acquisitions, $129 per parcel for the Bronx Parkway Extension, $59 per parcel for the Eastern State Parkway, $72 per parcel for Syracuse grade-crossing eliminations, and $56 per parcel for State-wide grade-crossing eliminations, are of limited significance.

MARGINAL LAND ACQUISITION USED ONLY RARELY

Marginal land acquisition, or "excess condemnation" as it is popularly called, is the acquisition of land and property in addition to the immediate physical needs of a highway improvement but having definite economic and protective value in connection with such improvement. Despite the adequate enabling legislation [17], the device has been used but rarely by the counties in the acquisition of lands for State highways, largely because its possibilities are still unknown and

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[17] See page 314 for the legal provisions relating to marginal land acquisition.

TABLE 4.—*Summary of number of parcels, area, and property costs for all purchased acquisitions under the agencies surveyed*

Population group and land use	Highways			Parkways						Grade crossings					
				Bronx Parkway Extension			Eastern State Parkway			Syracuse			State-wide		
	Number of parcels	Area	Property cost	Number of parcels	Area	Property cost	Number of parcels	Area	Property cost	Number of parcels	Area	Property cost	Number of parcels	Area	Property cost
		Square yards			*Square yards*			*Square yards*			*Square yards*			*Square yards*	
Grand total	1,746	5,693,714	$1,306,892	91	1,584,033	$349,294	91	16,133,591	$126,472	25	5,176	$92,275	365	556,962	$509,443
Rural	1,223	4,935,756	866,449	43	946,215	96,476	91	16,133,591	126,472	----	----	----	133	212,412	91,857
Suburban	466	605,120	408,585	24	388,508	125,690	----	----	----	----	----	----	68	70,785	94,029
Urban	57	152,838	31,858	24	249,310	127,128	----	----	----	25	5,176	92,275	164	273,765	323,557
Agricultural	1,161	5,212,069	832,574	16	432,610	21,587	88	15,915,714	113,971	----	----	----	124	384,806	71,447
Rural	914	4,663,407	653,239	16	432,610	21,587	88	15,915,714	113,971	----	----	----	91	191,710	43,690
Suburban	223	414,283	167,643	----	----	----	----	----	----	----	----	----	7	1,331	166
Urban	24	134,379	11,692	----	----	----	----	----	----	----	----	----	26	191,765	27,591
Residential	442	244,095	312,614	64	980,302	283,724	2	100,851	6,701	25	5,176	92,275	148	104,942	253,883
Rural	217	150,352	118,511	25	445,550	68,670	2	100,851	6,701	----	----	----	19	7,871	20,899
Suburban	200	87,246	178,119	18	285,795	90,326	----	----	----	----	----	----	44	56,995	66,326
Urban	25	6,497	15,984	21	248,957	124,728	----	----	----	25	5,176	92,275	85	40,076	166,658
All other land uses [1]	143	237,550	161,704	11	171,121	43,983	1	117,026	5,800	----	----	----	93	67,214	184,113
Rural	92	121,997	94,699	2	68,055	6,219	1	117,026	5,800	----	----	----	23	12,831	27,268
Suburban	43	103,591	62,823	6	102,713	35,364	----	----	----	----	----	----	17	12,459	27,537
Urban	8	11,962	4,182	3	353	2,400	----	----	----	----	----	----	53	41,924	129,308

[1] Includes commercial, industrial, semipublic, and public parcels.

untried. It is believed that the device can be utilized with particular effectiveness. Highway improvement often creates small remnants between the new and existing highways, of little use to their former owners. Columbia is practically the only county studied where extensive marginal lands were acquired and incorporated into the right-of-way. Extra costs were not incurred, since large payments for consequential damages would have been necessary had not the marginal lands been acquired.

The Grade Crossing Bureau does not acquire any more land than is required for the physical needs of an elimination. It seems that the marginal acquisition device could be utilized to good advantage in the grade-elimination program to avoid high consequential damages and the creation of remnants, for grade-crossing projects necessarily involve extreme changes of grade and severance injuries.

SCOPE OF THE INVESTIGATION BROAD

This investigation was undertaken to compare and appraise the various practices utilized in New York in the acquisition of lands for State highways, parkways,

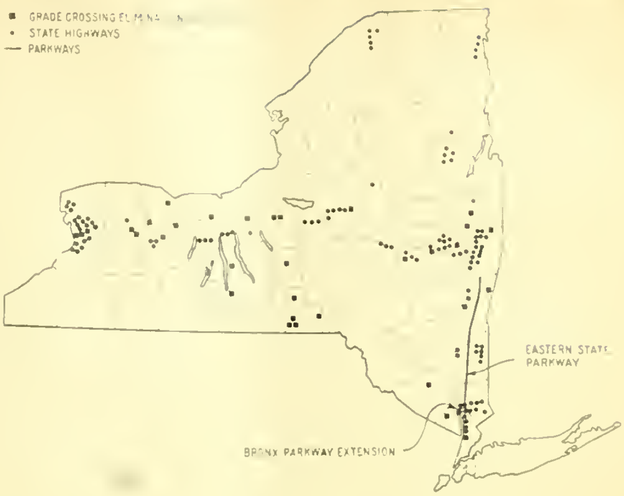

FIGURE 2.—GEOGRAPHIC DISTRIBUTION OF PROJECTS INVOLVING STATE HIGHWAYS, GRADE-CROSSING ELIMINATIONS, AND PARKWAYS.

TABLE 5.—*Summary of improvement and consequential damage costs for all purchased acquisitions under five agencies surveyed*

Population group and land use	Highways						Parkways											
							Bronx Parkway Extension						Eastern State Parkway					
	Total damage cost		Improvement damages		Consequential damages		Total damage cost		Improvement damages		Consequential damages		Total damage cost		Improvement damages		Consequential damages	
	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent
Grand Total	$991,191	100	$468,349	47	$522,842	53	$61,222	100	$45,687	75	$15,535	25	$9,613	100	$7,300	76	$2,313	24
Rural	669,204	100	264,587	40	404,617	60	12,324	100	6,787	55	5,537	45	9,613	100	7,300	76	2,313	24
Suburban	301,235	100	198,192	66	103,043	34	27,296	100	19,200	70	8,096	30						
Urban	20,752	100	5,570	27	15,182	73	21,602	100	19,700	91	1,902	9						
Agricultural	639,881	100	280,816	44	359,065	56	600	100			600	100	4,813	100	2,500	52	2,313	48
Rural	497,017	100	195,600	39	301,417	61	600	100			600	100	4,813	100	2,500	52	2,313	48
Suburban	136,830	100	84,689	62	52,141	38												
Urban	6,034	100	527	9	5,507	91												
Residential	223,953	100	114,059	51	109,894	49	58,976	100	44,187	75	14,789	25	4,800	100	4,800	100		
Rural	92,878	100	31,363	34	61,515	66	11,724	100	6,787	58	4,937	42	4,800	100	4,800	100		
Suburban	118,556	100	77,732	66	40,824	34	27,150	100	19,200	71	7,950	29						
Urban	12,519	100	4,964	40	7,555	60	20,102	100	18,200	91	1,902	9						
Other land uses [1]	127,357	100	73,474	58	53,883	42	1,646	100	1,500	91	146	9						
Rural	79,309	100	37,624	47	41,685	53												
Suburban	45,849	100	35,771	78	10,078	22	146	100			146	100						
Urban	2,199	100	79	4	2,120	96	1,500	100	1,500	100								

Population group and land use	Grade Crossings											
	Syracuse						State-wide					
	Total damage cost		Improvement damages		Consequential damages		Total damage cost		Improvement damages		Consequential damages	
	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent
Grand total	$73,750	100	$72,550	98	$1,200	2	$361,731	100	$172,279	48	$189,452	52
Rural							74,262	100	36,286	49	37,976	51
Suburban							61,336	100	20,640	34	40,696	66
Urban	73,750	100	72,550	98	1,200	2	226,133	100	115,353	51	110,780	49
Agricultural							59,080	100	20,286	34	38,794	66
Rural							36,381	100	12,660	35	23,721	65
Suburban							126	100			126	100
Urban							22,573	100	7,626	34	14,947	66
Residential	73,750	100	72,550	98	1,200	2	179,590	100	92,332	51	87,258	49
Rural							16,946	100	8,321	49	8,625	51
Suburban							40,386	100	8,429	21	31,957	79
Urban	73,750	100	72,550	98	1,200	2	122,258	100	75,582	62	46,676	38
Other land uses [1]							123,061	100	59,661	48	63,400	52
Rural							20,935	100	15,305	73	5,630	27
Suburban							20,824	100	12,211	59	8,613	41
Urban							81,302	100	32,115	40	49,157	60

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[1] This includes commercial, industrial, semipublic, and public parcels.

TABLE 6.—*Total number of parcels and area included in survey shown by population group and kind of development* [1]

Item	Total		Rural		Suburban		Urban	
	Number of parcels	Area	Number of parcels	Area	Number of parcels	Area	Number of parcels	Area
		Square yards		*Square yards*		*Square yards*		*Square yards*
State highways	2,072	6,560,799	1,389	5,410,770	608	985,598	75	164,431
Counties selected as representative [2]	1,308	3,864,978	769	3,119,074	506	738,786	33	7,118
Dutchess	175	1,309,005	160	1,290,682	15	18,323		
Erie	260	252,626	97	101,369	142	146,874	21	4,383
Niagara	43	104,528	38	102,553	5	1,975		
Rensselaer	153	233,088	84	148,077	69	85,011		
St. Lawrence	85	158,172	85	158,172				
Schenectady	172	258,845	17	20,860	155	237,985		
Warren	208	1,098,256	208	1,098,256				
Westchester	212	450,458	80	199,105	120	248,618	12	2,735
Counties selected on route basis [3]	764	2,695,821	620	2,291,696	102	246,812	42	157,313
Albany	73	123,757			73	123,757		
Clinton	46	363,145	46	363,145				
Columbia	70	644,093	70	644,093				
Livingston	45	188,927	40	185,492			5	3,435
Oneida	119	273,144	91	158,374	24	108,575	4	6,195
Saratoga	61	217,025	32	80,683			29	136,342
Schoharie	190	485,184	190	485,184				
Other counties [4]	160	400,546	151	374,725	5	14,480	4	11,341
Grade crossings	517	723,039	184	298,060	89	124,667	244	300,312
State-wide	469	713,492	184	298,060	89	124,667	196	290,765
Syracuse	48	9,547					48	9,547
Parkways	253	23,281,006	199	22,451,425	28	534,046	26	295,535
Taconic	153	21,289,927	153	21,289,927				
Westchester	100	1,991,079	46	1,161,498	28	534,046	26	295,535

[1] The majority of the data in this survey were abstracted between August and December 1940, with the exception of Syracuse grade-crossing data which were abstracted in February 1941.

[2] These counties primarily selected on basis of population density, highway mileage, and extent of development.

[3] To avoid bias in county selection, acquisitions on routes 5, 9, and 20 also were investigated. Only those counties having 45 acquisitions or more are listed separately.

[4] This includes the following counties on routes 5, 9, and 20 having less than 45 acquisitions—Cayuga, Essex, Genesee, Herkimer, Madison, Montgomery, Ontario, Otsego, and Seneca.

and grade-crossing eliminations, in terms of efficiency and adequacy. This necessitated the assembly of statistics and analysis of practices for five different land acquisition agencies, namely, the counties which acquired rights-of-way for State highways, the Taconic State Park Commission which secured lands for the Eastern State Parkway, the Westchester County Park Commission as agent for the State in the establishment of the Bronx Parkway Extension, the State Grade Crossing Bureau which purchased the lands necessary for State-wide grade-crossing eliminations, and the Syracuse Grade Crossing Commission which acquired lands for Syracuse grade-crossing eliminations.

The scope of survey varied with each of these organizations, as indicated in table 6 and figure 2. For the State highway analysis, eight representative counties were selected on the basis of highway development, land uses, topography, economic characteristics, and availability of the data. To augment this sample, acquisitions were studied on three through routes, i. e., State Route 5 and US 20, which traverse the State from east to west, and US 9 extending from north to south. A sufficiently large sample was obtained from acquisitions taking place between January 1936 and July 1940; but because there was marked activity on the three through routes between 1935 and 1940, an additional year was included for these data.

Analysis of State-wide grade-crossing eliminations included 31 representative projects involving 469 parcels acquired between 1928 and 1940, omitting all city projects. A sample of 48 acquisitions in Syracuse was later included. The investigation of parkway data included all the acquisitions for the Eastern State Parkway and all lands constituting the northern portion of the Bronx Parkway Extension. These two parkways are major State projects that traverse areas somewhat comparable to those traversed by State highway projects for which data are given. A grand total of 2,842 parcels involving 6,315 acres of land were investigated, including 2,072 State highway parcels of 1,356 acres, 517 grade-crossing-elimination parcels of 149 acres, and 253 parkway parcels of 4,810 acres. All parcels for which negotiations had not been consummated were excluded.

An outline of the primary methods of public land acquisition for highways, parkways, and grade-crossing eliminations in the State of New York is given in the following paragraphs.

THE HIGHWAY LAW [18]

Rights-of-way for State highways must be acquired by the counties and at their own expense. Pursuant to a request by the State, the county board of supervisors, through its right-of-way committee, negotiates for and secures options to purchase highway rights-of-way. Plat maps of the individual parcels to be acquired are furnished the counties without charge by the State. A right of entry by the State and county exists as soon as an agreement is executed. Options are exercised by the county pursuant to special resolutions, deeds are obtained and the awards are paid by the county treasurer. This procedure takes place when the owner and the county can agree on the settlement price. Where difficulties arise, one of the following modes of condemnation may be utilized.[19]

Commissioner of Appraisal System.—Any county in the State may use this method of forcible acquisition. Upon petition by the county attorney, pursuant to resolution of the board of supervisors and after notice to the owner, three disinterested freeholders are appointed by the county judge as commissioners of appraisal. Each commissioner is entitled to compensation of $25 a day and his necessary expenses. Upon the filing of the oaths of office of these commissioners, title to the lands described in the petition and by a map filed in the office of the county clerk vests in the county, whereupon a right of entry exists. After hearing all the evidence the commissioners make an award and file a report of their findings, which is binding unless one of the parties appeals therefrom. When the State highway department is informed that all necessary properties have been purchased or placed under condemnation, construction may be initiated.

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[18] Art. III, secs. 30-36, Highway Law.

[19] Lands for limited-access highways may also be acquired by the counties, in fee simple, either by purchase or by the exercise of the power of eminent domain as in the case of ordinary State highways.

Court or referee system.—Erie County is permitted to use a summary method of condemnation. After a copy of the petition and map of the parcel to be condemned are filed with the county clerk and notice has been given the owner and other interested parties the county may enter upon the lands. The proceedings are conducted in the supreme court or before an official referee designated by the court to hear the case and make the award. The court or referee views the premises, hears the testimony, and renders a decision which, upon application of the board of supervisors, can be made final within four months of its rendition. Any number of parcels of land on one or more highway improvements may be joined in the same proceeding.

THE STATE-WIDE GRADE CROSSING ELIMINATION ACT

The act applies to all highway-railroad crossing eliminations outside of the cities of New York, Buffalo, and Syracuse. Lands necessary for the elimination of railroad-highway grade crossings are acquired by the State through the Grade Crossing Bureau[20] of the Department of Public Works. After the necessity for the acquisition of lands for a particular grade-crossing elimination has been established, a map of the parcel and description is filed in the office of the Secretary of State and notice to that effect is given the Public Service Commission. Notice to the owner completes the appropriation, title passes to the State, and the State may enter upon the property and begin construction.[21] After careful appraisals are made, the State negotiates a settlement with the owner before actual occupation of the premises takes place. If settlement is not reached, the owner must file a claim for the value of his property in the State Court of Claims within 2 years of the appropriation.

The Constitution of New York authorizes the legislature to create a debt not exceeding $300,000,000 for grade-crossing elimination.[22] Originally, the cost of an elimination was to be shared 50 percent by the railroad company and 50 percent by the State, of which 10 percent was to be contributed by the minor governmental unit in which the elimination was situated. In 1929, the local units were charged with 1 percent, the State 49 percent, and the railroad 50 percent of the cost. These cost proportions were again changed in 1938, the railroad's share being reduced to an amount not to exceed 15 percent of the cost. If Federal funds are used for reconstruction of a highway-railroad grade separation structure on the State system, condemnation under the Highway Law is mandatory, and the summary appropriation method applicable ordinarily may not be used.[23]

THE SYRACUSE GRADE CROSSING ELIMINATION ACT

The act is an adaptation of the general Grade Crossing Elimination Act to urban uses.[24] The financing of eliminations follows the general act except that the administrative expenses of a special grade crossing commission, represented by special counsel, are city charges. The Public Service Commission determines the necessity of each undertaking and thereafter plans are filed with the city engineer upon approval of the common council. If the Syracuse Commission cannot purchase the necessary lands, it must resort to the machinery of the Condemnation Law, while claims for special additional damages must be made by the owner in the Court of Claims.

THE PARKWAY ACTS

Parks and parkways in New York are authorized in a number of separate laws, analyzed in the following paragraphs:

Taconic State Park Commission Act.—The Taconic State Park Commission[25] consists of five persons, appointed by the Governor, who receive only their necessary expenses. The commission has the power to establish State parks and parkways, including the Eastern State Parkway, and to acquire lands by purchase or by entry and appropriation under the provisions of the Conservation Law[26] or under the Condemnation Law. The Conservation Law sanctions appropriation, with the approval of the governor, of real property by filing a description with the Secretary of State. When the owner has been notified of such action, title to the property vests in the State, and within 2 years the owner may present his claims for the value of his property to the Court of Claims. Financing is done entirely by the State.

Westchester County Park Commission as agent for the State.—The Westchester County Park Commission consists of eight residents of Westchester County appointed by the board of supervisors,[27] and they receive only their necessary expenses. The commission, as agent for the State, has the power to establish and control State parks and parkways, including the Bronx Parkway Extension, in the county of Westchester. In the acquisition of lands for State parkways, the commission operates in the same manner as in the acquisition of lands for county purposes. The commission negotiates with property owners and takes options which must be approved by the board of supervisors.

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[20] The Grade Crossing Bureau also acquires lands and property for existing and new State institutions; for bridges and approaches built by the State under special acts of the legislature; for the Whiteface Mountain Highway under ch. 174, Laws of 1938; for the Little Falls Highway under ch. 698, Laws of 1936; for flood control purposes, under ch. 592, Laws of 1939; and for canal purposes under ch. 542, Laws of 1939.

[21] This right of immediate entry and possession is qualified by ch. 101, New York Laws of 1940, which specifies that the State may proceed against the owner or occupant of any lands only at or after the expiration of 30 days from the service of the copy of the description and plat maps and notice of the filing thereof in the office of the department of State; then, the State may proceed in the same manner as in the case of a tenant holding over after the expiration of his term without permission of his landlord, except that such action is to be preceded by 10 days' notice to quit.

[22] Art. 7, sec. 14 as amended in 1925.

[23] Ch. 375, Laws of 1939.

[24] Laws of 1928, ch. 825 enacted to revise the Laws of 1926, ch. 439.

[25] Created by ch. 198, Laws of 1925.

[26] Sec. 59.

[27] Ch. 292, Laws of 1922 amended by ch. 119, Laws of 1925, ch. 226, Laws of 1926; ch. 482, Laws 1927; ch. 441, Laws 1928, ch. 563, Laws 1929; ch. 312 and 561, Laws 1931.

In the event compulsory acquisition is necessary, the commission proceeds with condemnation as under the Highway Law. The Bronx Parkway Extension was financed completely by the State.

THE CONDEMNATION LAW

In addition to the provisions indicated under the Highway Law, the Condemnation Law specifies that costs of the proceedings be awarded as follows:[28] (*a*) If the award exceeds the offer, an allowance of $65 is made for court costs and the court may grant an additional amount not exceeding 5 percent of the award. (*b*) The property owner is allowed interest at the rate of 6 percent from the institution of proceedings to date of award and 4 percent thereafter to the date of payment.

MARGINAL LAND ACQUISITION

New York is one of the few States which sanctions the use of the "excess condemnation" device by organic law. Article I, section 7, of the State Constitution provides:

> The Legislature may authorize cities and counties to take more land and property than is needed for actual construction in the laying out, widening, extending, or relocating parks, public places, highways or streets; provided, however, that the additional land and property so authorized to be taken shall be no more than sufficient to form suitable building sites abutting on such park, public place, highway or street. After so much of the land and property has been appropriated for such park, public place, highway or street as is needed therefor, the remainder may be sold or leased.

It was introduced as part of the organic law in 1913, and by a subsequent amendment in 1927, it was enlarged to include counties as well as cities.

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[28] Sec. 16.

TESTS OF CONCRETE CONTAINING FLORIDA "OJUS" ROCK

BY THE DIVISION OF PHYSICAL RESEARCH AND TESTS, PUBLIC ROADS ADMINISTRATION

Reported by **W. E. Grieb**, Associate Highway Engineer

THE PURPOSE of this investigation was to determine whether aggregate made from the Florida coral formation known as "Ojus" rock is as satisfactory as natural sand and trap rock for use in reinforced concrete exposed to the action of sea water. Because of its quite porous nature, the ability of Ojus rock to protect imbedded reinforcing steel against corrosion by action of salt water has been questioned. The tests were designed to throw light on this particular question and were made by subjecting concrete specimens in which plain steel rods were imbedded to alternate immersions in synthetic sea water followed by drying in air. The amount of protection afforded the steel was determined by breaking open specimens from time to time and noting the condition of the rods.

TESTS MADE WITH THREE COMBINATIONS OF AGGREGATE AND TWO CEMENTS

Three combinations of aggregate were used in the tests, as follows:

(*a*) Trap rock with Lake Wales natural sand as fine aggregate,

(*b*) Ojus rock with Ojus rock screenings as fine aggregate,

(*c*) Ojus rock with Lake Wales natural sand as fine aggregate.

Each of the above combinations was used with two portland cements, a cement relatively high in computed amount of tricalcium aluminate (C_3A) and a cement containing virtually no tricalcium aluminate. Both cements passed the A. S. T. M. specifications for portland cement in effect at the time these tests were started. In this report the first cement is identified as type I and the second as type II. Complete chemical analyses of the cements are not available. However, tests on other cements from the same sources indicate that the two cements are properly classified as to type.

The gradings and physical properties of the aggregates are given in table 1 and data for the concrete mixes are given in table 2.

For each of the six combinations of materials from thirteen to fifteen 6- by 12-inch concrete cylinders were made, the number depending upon the amount of material available. All specimens were given an initial curing of 28 days in moist air. Three cylinders from each combination were then tested for compressive strength. The remaining specimens were divided into three groups, as follows:

Group A.—Five specimens were cast with each combination of materials, each containing two ½-inch round plain steel rods, 8 inches long. One rod was imbedded in the center of the specimen and equidistant from the ends. The other rod was imbedded half way between the center and the outside and also equidistant from the ends. One rod in each specimen had approximately 3 inches of cover as compared to about 1½ inches of cover for the other. These specimens were alternately immersed in synthetic sea water for 24 hours and dried in laboratory air for 24 hours. They were examined periodically to determine if rust was forming on the rods.

Group B.—Three specimens were cast with each combination of materials, each containing two ½-inch round plain steel rods 8 inches long and placed in the specimens in the same manner as those in group A. These specimens were alternately immersed in tap water for 24 hours and dried in laboratory air for 24 hours. They were examined along with the specimens in group A to determine whether rusting was due to sea water.

Group C.—From two to four specimens were cast with each combination of materials. These specimens contained no steel and were treated in the same manner as the specimens in group B. They were tested for compressive strength when the final inspection of the specimens in groups A and B was made.

After 200 alternations of wetting and drying, two specimens for each combination of materials from group A and one specimen for each combination from group B were examined for evidence of corrosion of the steel. The specimens were broken open and the steel removed. A slight amount of rust was found on one of the rods taken from a specimen from group A containing type II cement, Lake Wales sand, and Ojus rock. The other specimens appeared to be unaffected.

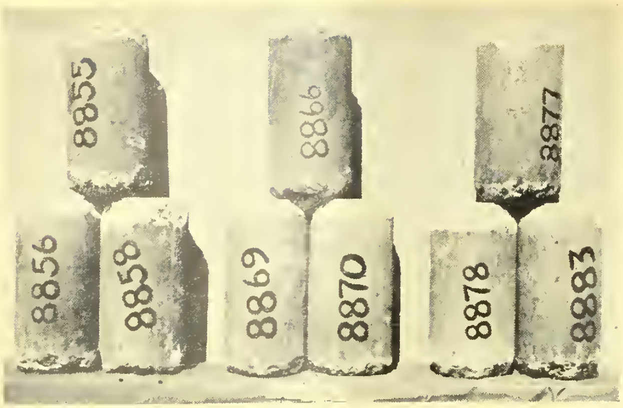
After 500 alternations, all of the remaining specimens of groups A and B were weighed, the loss in weight

TABLE 1.—*Tests of aggregates*

	Ojus rock	Trap rock	Ojus sand	Lake Wales sand
Sieve analysis:				
Percent retained on—				
¾-inch sieve	0	0		
½-inch sieve	67	67		
No. 4 sieve	100	100	0	0
No. 8 sieve			9	1
No. 16 sieve			31	10
No. 30 sieve			45	46
No. 50 sieve			67	80
No. 100 sieve			94	99
Fineness modulus			2.46	2.36
Percent absorption	4.59		1.5	0.3
Bulk specific gravity	2.27	2.91	2.56	2.63
Weight per cubic foot, (dry loose), pounds	79	98	96	105

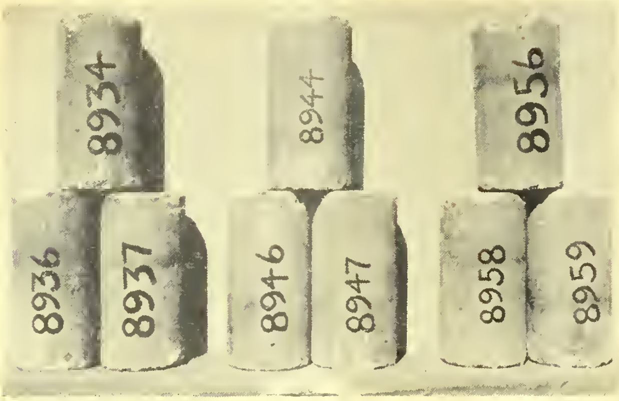
TABLE 2.—*Data on concrete proportions*

Material combination			Proportions by weight	Cement factor	Water-cement ratio	Slump
Type cement	Fine aggregate	Coarse aggregate				
				Sacks per cu yd.	*Gallons per sack*	*Inches*
I	Lake Wales	Trap	1:2.56:3.46	5.71	4.6	5¼
I	Ojus	Ojus	1:1.97:2.55	6.12	5.7	4½
I	Lake Wales	do	1:2.25:3.22	5.46	5.5	4½
II	do	Trap	1:2.56:3.46	5.71	4.6	5½
II	Ojus	Ojus	1:1.97:2.55	6.12	5.7	5
II	Lake Wales	do	1:2.25:3.22	5.46	5.5	4¾



Lake Wales sand and trap rock — Ojus sand and Ojus rock — Lake Wales sand and Ojus rock

FIGURE 1.—CONDITION OF SPECIMENS MADE WITH TYPE I CEMENT AFTER IMMERSION IN SEA WATER AND DRYING 850 TIMES.



Lake Wales sand and trap rock — Ojus sand and Ojus rock — Lake Wales sand and Ojus rock

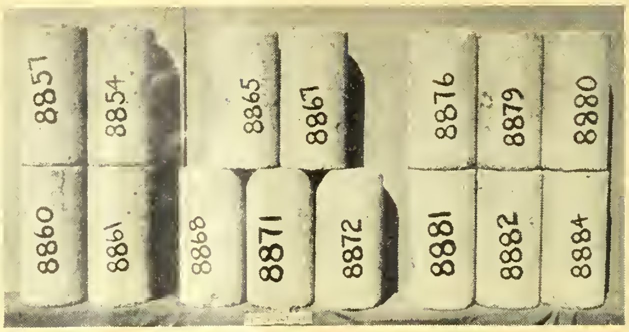
FIGURE 2.—CONDITION OF SPECIMENS MADE WITH TYPE II CEMENT AFTER IMMERSION IN SEA WATER AND DRYING 850 TIMES.

recorded and the specimens examined for evidence of rust on the surface of the concrete. These observations were repeated at the end of 600 and 700 alternations.

At 850 alternations, one specimen from each combination in groups A and B was broken open and the steel examined. Considerable corrosion was noted in all specimens of group A. It was decided to discontinue the treatment and to make the final comparisons on the basis of the action at the end of 850 alternations of wetting and drying. At this point the specimens were approximately 7 years old.

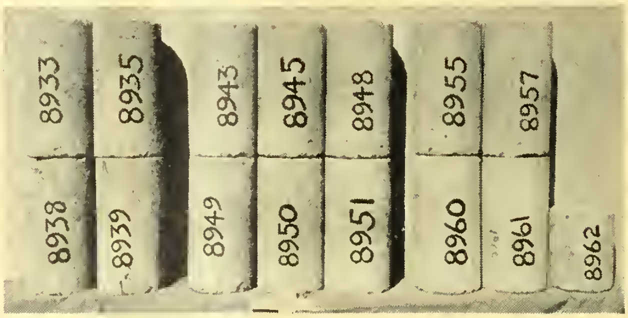
EFFECT OF IMMERSION IN SEA WATER ON THE CONCRETE

The losses in weight of the concrete specimens after 500 and 850 alternations of immersion in sea and in tap water followed by drying in air are shown in table 3. Each result is the average of tests on three specimens. Photographs of the individual specimens after 850 alternations are shown in figures 1 to 4, inclusive. In each figure, the specimens are arranged in three groups, according to aggregates used. The grouping in the photographs is in the same order as in the table, thus facilitating comparisons. In the case of specimens stored in tap water, cylinders from both groups B and C are shown.



Lake Wales sand and trap rock — Ojus sand and Ojus rock — Lake Wales sand and Ojus rock

FIGURE 3.—CONDITION OF SPECIMENS MADE WITH TYPE I CEMENT AFTER IMMERSION IN TAP WATER AND DRYING 850 TIMES.



Lake Wales sand and trap rock — Ojus sand and Ojus rock — Lake Wales sand and Ojus rock

FIGURE 4.—CONDITION OF SPECIMENS MADE WITH TYPE II CEMENT AFTER IMMERSION IN TAP WATER AND DRYING 850 TIMES.

In comparing the weight losses given in table 3 it should be borne in mind that the large amount of handling required in wetting and drying the specimens probably produced some surface abrasion, particularly at the edges. It is believed that this probably accounts for most of the losses shown for the specimens stored in tap water and also for those containing type II cement and stored in sea water. The fact that the specimens made with trap rock showed somewhat

TABLE 3.—*Weight losses of concrete specimens after alternate wetting and drying*

IMMERSION IN SEA WATER

Material combination			Percent loss in weight after	
Type of cement	Fine aggregate	Coarse aggregate	500 alternations	850 alternations
I	Lake Wales	Trap	3.0	3.7
I	Ojus	Ojus	3.7	4.8
I	Lake Wales	do	3.8	5.3
II	do	Trap	.4	1.0
II	Ojus	Ojus	2.6	3.2
II	Lake Wales	do	1.5	2.1

IMMERSION IN TAP WATER

Type of cement	Fine aggregate	Coarse aggregate	500 alternations	850 alternations
I	Lake Wales	Trap	0.9	1.5
I	Ojus	Ojus	1.8	2.7
I	Lake Wales	do	1.8	2.5
II	do	Trap	.4	1.2
II	Ojus	Ojus	1.2	2.8
II	Lake Wales	do	1.1	2.6

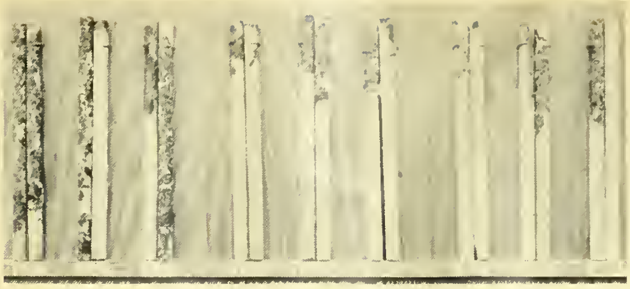

Lake Wales sand and trap rock — Ojus sand and Ojus rock — Lake Wales sand and Ojus rock

FIGURE 5.—CONDITION OF RODS IN SPECIMENS MADE WITH TYPE I CEMENT AFTER IMMERSION IN SEA WATER AND DRYING 850 TIMES.

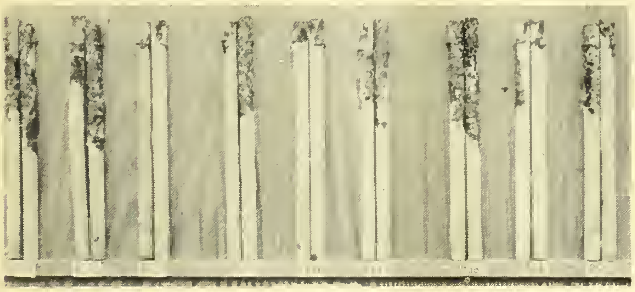
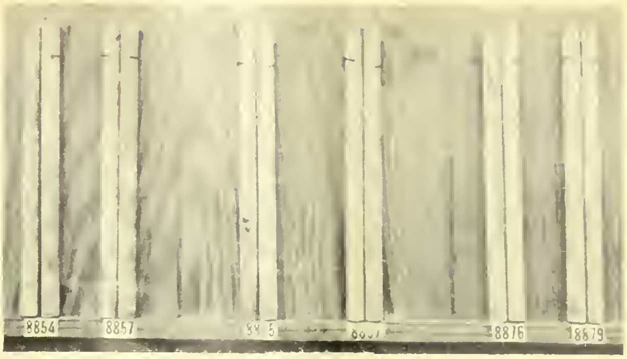
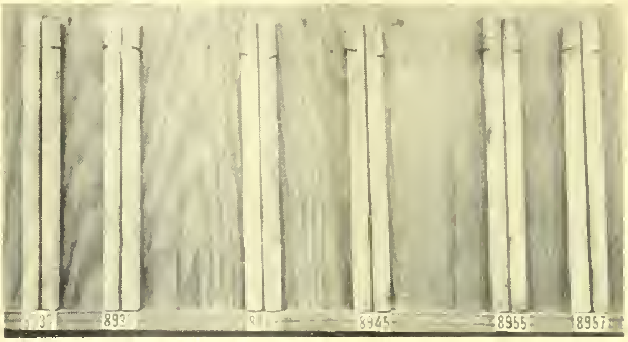

Lake Wales sand and trap rock — Ojus sand and Ojus rock — Lake Wales sand and Ojus rock

FIGURE 6.—CONDITION OF RODS IN SPECIMENS MADE WITH TYPE II CEMENT AFTER IMMERSION IN SEA WATER AND DRYING 850 TIMES.



Lake Wales sand and trap rock — Ojus sand and Ojus rock — Lake Wales sand and Ojus rock

FIGURE 7.—CONDITION OF RODS IN SPECIMENS MADE WITH TYPE I CEMENT AFTER IMMERSION IN TAP WATER AND DRYING 850 TIMES.



Lake Wales sand and trap rock — Ojus sand and Ojus rock — Lake Wales sand and Ojus rock

FIGURE 8.—CONDITION OF RODS IN SPECIMENS MADE WITH TYPE II CEMENT AFTER IMMERSION IN TAP WATER AND DRYING 850 TIMES.

lower weight losses than those made with Ojus rock may be due to the superior hardness of the trap rock aggregate. These differences in weight losses are about the same regardless of the type of exposure.

However, the specimens containing type I cement stored in sea water show considerably higher losses than the corresponding specimens in tap water indicating that, entirely independent of the possible effect of surface abrasion caused by handling, exposure to sea water has had some effect on the specimens containing the type I cement.

EFFECT OF IMMERSION IN SEA WATER ON THE STEEL

The extent and character of the corrosion that developed on the steel rods during 850 immersions in sea water are given in table 4 and are shown by photographs of the individual bars in figures 5 and 6. In each figure the bars are arranged in three groups of 6 bars each, according to aggregates used in making the cylinders. In each pair the left hand bar was taken from the side of the specimen, the right hand bar from the center. The bars are arranged with the corroded portion at the top without regard to their original positions in the concrete cylinders.

Table 4 and figures 5 and 6 show wide variations in the amount and character of the corrosion developed. For instance, in the first group (type I cement with Lake Wales sand and trap rock), the center rod from one of the specimens showed only 2 percent corrosion as compared to 70 percent for the outside rod. These individual discrepancies are no doubt due to variations in the degree of protection afforded by the concrete adjacent to the steel. This in turn was probably influenced by accidental variations in density. It is significant that, on the average, the concrete containing

TABLE 4.—*Condition of steel after 850 alternations of immersion in sea water and drying*

Material combination			Steel from side of specimens		Steel from center of specimens		Surface affected average for group
Type of cement	Fine aggregate	Coarse aggregate	Surface affected	Condition of steel	Surface affected	Condition of steel	
			Percent		*Percent*		*Percent*
I	Lake Wales	Trap rock	70	Deep pitting	80	Deep pitting	55
			70	do	2	do	
			30	do	75	do	
I	Ojus	Ojus rock	15	do	2	Slight pitting	17
			20	do	25	Deep pitting	
			30	do	10	do	
I	Lake Wales	do	15	Pitting	1	do	13
			5	Rust	15	do	
			40	Deep pitting	0	O. K	
II	do	Trap rock	15	do	10	Deep pitting	21
			25	do	45	do	
			2	Rust	1	Slight pitting	
II	Ojus	Ojus rock	2	Deep pitting	15	Deep pitting	12
			2	Pitting	3	do	
			20	Deep pitting	30	do	
II	Lake Wales	do	35	do	35	do	17
			10	Pitting	2	do	
			15	Deep pitting	3	do	

Table 5.—*Crushing strength tests*

Material combination			Crushing strength, pounds per square inch	
Type of cement	Fine aggregate	Coarse aggregate	28 days [1]	850 alternations [2]
I	Lake Wales	Trap rock	2, 860	3, 430
I	Ojus	Ojus rock	3, 070	4, 140
I	Lake Wales	do	2, 880	4, 340
II	do	Trap rock	3, 300	4, 140
II	Ojus	Ojus rock	3, 240	4, 110
II	Lake Wales	do	3, 580	4, 340

[1] Average of three specimens.
[2] Average of from two to four specimens approximately 7 years old.

the Ojus rock afforded as much or even somewhat greater protection than the concrete containing Lake Wales sand and trap rock. As a matter of fact the first group shows by far the greatest average attack. The apparent superiority of the type II cement in resisting sea water attack on the surface of the concrete does not hold for the protection of the steel. The type of cement appears to make little difference.

Figures 7 and 8 show the condition of the rods from the specimens immersed in tap water. There is no evidence of corrosion of the steel other than very slight rusting of one or two of the bars.

The results of strength tests after 28 days moist curing and after 850 alternations of immersion in tap water for 24 hours followed by air drying for 24 hours are shown in table 5. The results are not particularly significant except in showing that there was no retrogression in strength during the 7-year period covered by this test.

CONCLUSIONS

The results of these tests indicate:

(1) That concrete containing Ojus rock, either as total aggregate or in combination with Lake Wales sand, is somewhat less resistant to sea water attack than similar concrete containing Lake Wales sand and trap rock.

(2) That concrete containing the type I cement used in these tests is somewhat less resistant to sea water attack than similar concrete containing the type II cement used in the tests.

(3) That concrete containing Ojus rock, either alone or in combination with Lake Wales sand, is as effective in protecting reinforcing steel against sea water attack as is concrete containing Lake Wales sand and trap rock.

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